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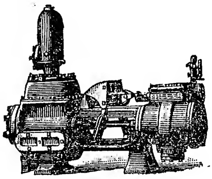

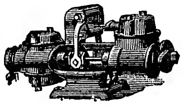

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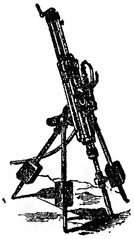

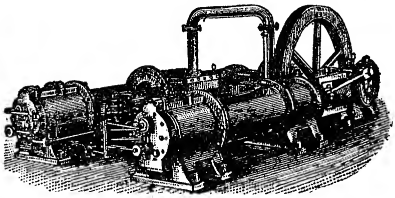

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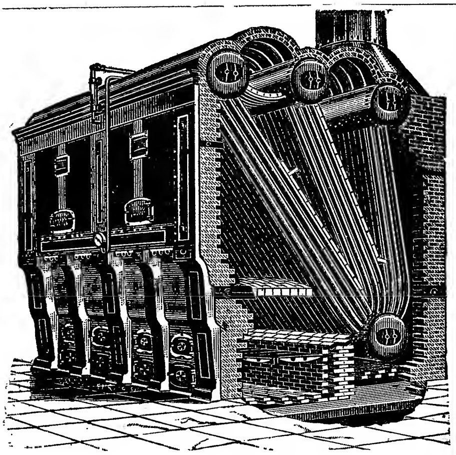

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PUBLISHED BY THE

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CHAMPAIGN, ILLINOIS,

1892-93.

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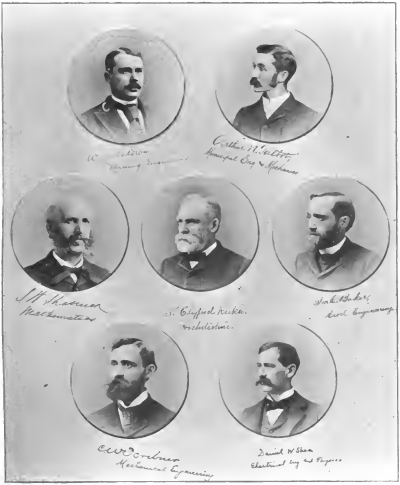

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College of Engineering, University of Illinois.

THE TECHNOGRAPH.

No. 7. UNIVERSITY OF ILLINOIS. 1892–93.

DEEP PILE FOUNDATIONS IN CHICAGO.

BY GEN. WM. SOOY SMITH, CONSULTING CIVIL ENGINEER, CHICAGO.

[In the preceding volume of THE TECHNOGRAPH was an article by General Smith, an able civil engineer who has large experience with foundations for heavy buildings, on the building problem in Chicago, in which a new plan for deep foundations was proposed. In the following article General Smith gives an account of the method now being employed by himself in sinking one of these foundations.—EDITORS.]

The acceptance, by the Board of Directors who are now erecting a building for the Chicago Public Library, of a design for deep foundations for that structure, marks an era in the history of heavy building construction in Chicago. It has for some time been evident that the system of independent steel footings, spread or floated upon the unstable clay which underlies the business district, is far from satisfactory. At many points in the city small areas of the clay surface have been tested by imposing loads to a maximum of three tons per square foot. It has been found in this way that as the load is increased, a proportional settlement takes place with all weights under two to two and one-half tons per square foot, and that this settlement apparently ceases in a short time. With rapid sinking and with loads above two to two and one-half tons per square foot, depending upon the location of the test, the ratio of settlement becomes greater with sometimes a well marked "point of failure," though in every case, with the maximum load thus applied, a point has soon been reached where the settlement apparently ceases. From information obtained from these tests, it has been the custom among architects to assume 3 000 pounds per square foot as the load to be safely borne, and to depend upon a careful calculation of dead and live loads to insure an equal amount of settlement for each pier. An inspection of the older buildings constructed upon this plan shows that where the full load was imposed, there has been a constant subsidence since the initial settlement, usually imperceptible except by observation at long intervals, in one case as much as $\frac{1}{16}$ of an inch per month. In many of our heavy buildings there is to be found abundant evidence that this sinking has not taken place uniformly over the area covered by the buildings.

These facts make it evident that the system of spread foundations must be abandoned. At most points in the district to which tall building construction has been confined, there is found at about 45 feet below city datum a very compact bed of clay containing considerable shale gravel which overlies the bed rock,—the latter being from 90 to 110 feet below city datum. The foundations of at least three of our later buildings reach this lower clay bed by means of piles sawed off and capped just below present water level. At the Grand Central Passenger Station these piles sustain loads of 24 tons each with no settlement to date. In driving test piles at the Schiller Opera House it was found that when the point of a pile had entered the hard clay, the succeeding blows produced but little effect. In one case it required forty-two strokes of the steam hammer to produce one inch of penetration. This lower clay bed, then, is practically as unyielding as the rock itself.

To secure a foundation in which there shall be no settlement, to provide for the possibility of future deep drainage or disturbance of the upper strata by excavations near the building, to render possible any change in distribution of loads which may become necessary or desirable, and to utilize area to a depth greater than that reached by ordinary footings, the following plan was devised, and the progress of the work on the Public Library Building has demonstrated that a perfectly satisfactory solution of the problem has been reached. For this building trenches were excavated upon the lines of the various walls, of a width to permit the necessary shoring and to include three rows of piles to be driven three feet between centers. The character of the upper clay stratum was shown by the fact that it crowded through the smallest crack in the sheet piling almost as if it were quicksand. The depth of excavation permitted sawing and capping the piles at a point such that the highest level of timber used in the grillage is fourteen feet below city datum, which is about lake level. From this grillage rubble masonry is carried up to the neat masonry courses of the basement.

While the work was in progress, a test was made to determine the bearing power of four of the piles as they had been driven in the trench. The piles are of Norway pine. The driving was done with a steam hammer weighing in all 8 300 pounds, the hammer alone weighing 4 500 pounds, and delivering fifty-four blows per minute with a stroke of 42 inches. The last 20 feet of driving was by means of an oak follower. The penetration was about one inch for six blows. The piles were driven 2 feet 6 inches between centers. The bearing power of the piles was tested by constructing a platform on top of four piles and loading it with pig iron. Levels were carefully taken on each of the four piles. They stood four days with a loading of 5.5 tons per pile, eight days with a loading of 37.3 tons per pile, and ten days with a load of 50.6 tons per pile, all without appreciable settlement—at most, not as great as one hundredth of a foot. If 250 pounds per square inch be estimated for point resistance, the average frictional resistance will be about 3.2 pounds per square inch of side surface of pile, or about 432 pounds per square foot.

For an ordinary pile, 7 inches at top and 14 inches at butt, driven 45 feet, the frictional resistance is 59 000 pounds, and the point resistance 6 000 pounds; hence, the total earth resistance is 65 000 pounds, with a factor of safety of 5 to 6. The resistance of the pile, considered as a column, is 65 000 pounds, with a factor of safety of 3 to 4. The minimum safe load, therefore, is 33 net tons.

This form of foundation commends itself from an economical standpoint.

A DEEP PILE FOUNDATION.

By C. J. Mitchell, '91, Asst. Civil Engineer I. C. R. R.

The new Illinois Central Railway Passenger Station and Office Building at 12th street, Chicago, is 180 by 220 ft.; and consists of the office portion 9 stories high, the tower 13 stories, and the station 3 stories high. The train shed with connection to the station covers 8 tracks, and is 680 ft. long.

Borings taken on the site showed from 10 to 20 feet of miscellaneous rubbish which had been dumped there immediately after the great fire. Below this made ground were found irregular layers of stiff blue clay and quicksand. Rock was not struck until a depth of more than 60 feet was reached. Soil of this character could hardly be trusted to carry the ordinary Chicago type of independent pier foundation, more especially since, owing to the nature of the building and the arrangement of the tracks, trains running near and under a portion of it would have a tendency to cause a settling of the foundations.

It was determined to use a deep pile foundation the peculiar feature of which consists in driving long piles to a firm bearing and sawing the tops off "at a level below that of any conceivable deep drainage system that many become desirable or necessary in the city."

Naturally the rather uncertain quality of the soil made it necessary to use a large number of piles, about 1 700 being driven. The piles were arranged in groups or clusters under each column. In the head-house portion consisting of three stories above the passenger tracks, these groups were of three sizes; viz., 8, 13 and 16 piles each. In the office portion (9 stories high) the groups contained 20 to 42 piles each; while under the tower (13 stories) one of the corner clusters has 73 piles. The piles in the groups were sometimes arranged on the square, lining both ways; but usually alternating in rows 18 inches apart making the distance between centers 25½ inches. This is as close as these piles could be driven, and sometimes in hard ground the position of the tops when cut off bore no resemblance to that in which the tips were started, while in a few clusters the piles did not vary more than 1 inch from where they were started.

The piles varied from 11 to 16 inches in diameter at the butt end, and were from 40 to 60 feet in length, averaging about 51 feet. It was specified that the piles should have a uniform taper, and no crooks, bends, knots, windshakes, etc.; but before driving care was taken to reject those having too much or not enough taper and also those having bends in two directions. 32 % of the piles were black gum, 22 % pine, 7 % basswood, 21 % oak, 15 % hickory, with a few maple and elm. A cast iron cap was used in driving, but in spite of this 8 % of the heads were crushed or split. The pine piles had the poorest driving record, the heads of 12½ % of them being crushed and 5 % broken; the gum came next 7 % being crushed and 0.6 % broken; the oak had 5 % crushed and 0.8 % broken; the hickory had 3 % crushed and none broken; 8 % of the basswood piles had the heads crushed. In several of the oak piles, the sap wood was separated from the heart, the cores being driven through the shells. All of the piles were pointed to 4 inches square.

The drivers used, three in number, were of the usual pattern of drop hammer, the weights of which were 2 800, 3 200 and 3 800 pounds respectively. The driver having the heaviest hammer made the best record; viz., 26 piles driven home to a depth of at least 60 feet below the surface. However there is some doubt as to whether this result was due to the greater weight of the hammer or to the greater ability of the crew of the driver.

The fall of the hammer at the last blow was usually 35 to 40 feet, though when driving close to a dwelling a fall of 25 feet was used with about as good success. The penetration at the last blow averaged about 3 inches, though a large number went less than 1½ inches. The cap prevented much loss by brooming of the heads.

The distance to which the vibration due to driving was felt varied with the height of fall, the character of the soil, and the spacing of the piles. The vibration was easily felt at a distance of 400 feet, while the effect at 75 feet was quite marked. However, in doing instrumental work it was sometimes found that the vibration within 25 feet of the driver was less severe than at several times that distance.

One curious effect that the driving of piles had on adjacent objects was observed when a group of sixteen piles was driven about 15 feet from a group of eight which had been sawed off to a uniform height and had waling pieces drifted on; the latter was raised 4 inches next to the driver and 1 inch on the opposite side. Under similar conditions when sixteen piles were driven about 15 feet from a finished pier consisting of eight 47 ft. piles, 2½ feet of grillage and concrete, and 12 feet of stone masonry, the pier raised ⅝ inches on the side next to the driver, but on the farther side (32 inches distant) it remained at the original elevation. Two weeks later this pier was tested and the high side was found only $\frac{1}{4}$ inch too high, while the other side had not changed. Again, in a group of seventy-two piles a spike was placed in the head of the first pile driven, and elevations were taken daily; the first two days the pile sunk $\frac{1}{2}$ inch, then rose steadily until fifty piles had been driven, when it was 3 inches above the first height, the greatest rise in one day being $\frac{3}{4}$ of an inch. This pile was 55 feet long, of which 45 feet was in the ground.

The piles were usually driven in groups till the tops of all were below the leads; then the driving was completed by means of a follower. Water was kept running around the pile at the surface when the pile was being driven, which seemed to help the driving to a considerable extent. After the piles had been driven, the tops were sawed off to a uniform height of 3 feet below datum in order that all timber should be below low water. As this was from 10 to 14 feet below the surface, it was necessary to excavate and sheathe the pits for the foundations and also to keep pumps running.

In excavating, the old pile trestle work up on which the Central formerly entered the city was uncovered in several places. It had been in the ground at least 21 years; but that portion which was below water was perfectly sound and was perhaps a trifle more dense than when first used, although the portion of the same piles above water had almost rotted away.

After the piles were cut off, the earth was excavated 18 inches below their tops and rich Portland cement concrete was tamped in even with the tops of the piles. Oak caps 12x12 inches were fastened on by a drift bolt in the center of each pile, and the space between the timbers was filled with concrete ; on top of this came the masonry work. Under the head-house, the masonry work on each pier consisted of 10 feet 6 inches of limestone and a plinth of Bedford limestone 2 feet thick in two courses about 12 inches each. All joints were about 1 inch thick. Under the office building there was 8 feet 6 inches of limestone, 2 feet of sandstone and 2 feet of granite, the granite being the plinth.

The stone was all set by a carriage running on tracks placed on either side of a row of piers ; hooks were used to handle all the foundation stone. The top stone was brought to grade by laying a bed of mortar $\frac{1}{4}$ to $\frac{3}{8}$ inches too thick and hammering the stone down to within 1-16 inch of the right height. The stones, even the heavy granite block 8 feet by 4 feet by 2 feet, were brought to center lines within $\frac{1}{4}$ inch without trouble.

A BIT OF EXPERIENCE WITH PILES IN CHICAGO.

BY SIMEON C. COLTON, '85, CIVIL ENGINEER, CHICAGO.

When driving the piles for the foundation of the Edison Light Co.'s Power House, corner of Harrison street and Chicago river, the representative of the architect of the building expressed a doubt as to the efficiency of a steam pile hammer. To settle this point three test piles were driven as described below.

Pile No. 1, 50 feet long, was driven with a drop hammer weighing 3 600 pounds. The sinking was as follows:

8 feet 0 inches in 10 blows of 6 feet each.
6 " 0 " " 10 " " 6 " "
6 " 6 " " 10 " " 10 " "
2 " 3 " " 4 " " 15 " "
4 " 0 " " 10 " " 15 " "
3 " 5 " " 10 " " 20 " "
1 " 9 " " 10 " " 20 " "
1 " 7 " " 10 " " 20 " "

Bearing Power by Baker's formula* = 20 tons.
" " " Engineering News formula† = 25 tons.

Pile No. 2 was driven with a steam hammer, the total weight of the hammer being 8 100 pounds and that of the striking weight 3 500. The stroke was 32 inches. The sinking was:

5 feet in 15 blows.
5 " " 19 "
3 " " 28 "
2 " " 31 "
1 " " 19 "
1 " " 20 "
1 " " 25 "
1 " " 20 "
0.62 " " 19 "

Bearing Power by Baker's formula=10.0 tons.
" " " Engineering News formula=6.7 tons.

Pile No. 3 was driven with same hammer as No. 2. The rate of sinking was:

8 feet by weight of hammer alone.
12 " in 23 blows.
3 " " 18 "
3 " " 27 "
3 " " 40 "
2 " " 40 "
0.5 " " 10 "

Bearing Power by Baker's formula = 7.0 tons.
" " " Engineering News formula = 6.2 tons.

After standing over night the sinking of pile No. 3, under the drop hammer, was:

0.4 inch for fall of 2 feet 8 inches.
1.6 " " " " 15 " 0 "
1.7 " " " " 20 " 0 "

---

*Baker's Masonry Construction, p. 239.
†Engineering News, Vol. 22, p. 373, (Oct. 19, 1889.)

The last blow of the steam hammer moved this pile 0.6 inch, while the same fall of a drop hammer having almost the some weight gave a penetration of only 0.4 inch. However the last penetration was after the pile had stood over night. The penetration of 1.7 inches for a fall of 20 feet shows pile No. 3 (driven with a steam hammer) to be better driven than No. 1 (driven with a drop hammer), which sunk 1.9 inches under a fall of 20 feet.

After standing over winter it became necessary to remove these test piles. In attempting to lift them a 1⅝ inch chain was twice broken. Finally a vertical lifting strain of 40 tons was put upon each of the piles for a considerable time, with a sort of dancing motion, without "fazing" any one of them. While trying to pull pile No. 1, a water-jet pipe was pushed down 35 feet alongside of the pile, but did not seem to effect it in the least. Each was then driven about 4 feet further, when they struck hard pan and could not be driven further with a 4 200 drop hammer.

The foundation was completed by sawing off the piles and constructing a layer of concrete 2 feet thick over their heads. The foundations of the walls are separated from the engine foundations, by a clear space of 12 inches, so that any settlement of one will not affect the other.

BRICKMAKING ON SOUTH AMERICAN RAILWAYS.

P. Mogensen, '94, Civil Engineering Course.

When railroads are constructed in territories where the bullock cart and the mule wagon have (as far as transportation is concerned) held undisputed sway for centuries, and when substantial inducements are offered the constructors either in the form of a government guarantee of from five to seven per cent. interest on the capital invested, or the certainty that a lucrative passenger and freight traffic is awaiting the completion of the work, and perhaps the possibility of competition has been removed by statutory enactments, it will generally be found neither profitable nor convenient to erect temporary structures such as trestles where waterways have to be bridged and wooden sheds where station houses are required.

Such are, or at least have been, in a large measure the conditions attending the construction of railways in the Argentine Republic and other South American countries bordering on the Atlantic. As it is always desirable to complete a line in the shortest time possible and to have the masonry work in bridges and station houses going on at the same time the roadbed is being built and the track laid, it often becomes necessary to have large quantities of bricks delivered in places far out of reach of established brick factories. This circumstance has called into existence, or properly speaking, revived the trade of making bricks in the field, the more important features of which it is my present purpose to describe briefly.

In selecting a site for a temporary brick field two things are primarily sought for—abundance of loam and ample supply of fresh water. The latter is indispensable but the former, though very desirable because of its easy preparation, is not an absolute necessity.

When the loam only is used it is dug out and spread in layers in a pit ; each layer is thoroughly saturated with water, care being taken to break up the earth as much as possible in the operation. It is left in this state for a day or two when it is again treated much in the same manner as before. This operation, which is called tempering, has the effect of making the clay more homogeneous, and as a result the bricks are better able to withstand the drying and burning processes without cracking. The earth is next worked to and fro with a hoe and, when necessary, is further disintegrated with a pounder until the required degree of plasticity has been obtained, and it is then ready for moulding.

When the material is refractory clay or " joint clay " with little or no black soil, the tempering is more laborious and expensive; and though the methods employed are essentially the same as those described aboye, it requires a much longer time, often a period of from one to three months. It frequently happens that urgent need for brick makes it necessary to begin moulding before the tempering is complete. In such cases more care and time must be given to drying and to burning than would otherwise be required.

In districts devoted to grazing purposes and where horses are plentiful a somewhat peculiar kind of "pug mill" is in general use. A round pit about 3 feet deep and 40 feet in diameter is dug in the ground and paved with bricks set on edge. In this pit the clay is placed in layers and well watered; on the top is sometimes added a layer of short straw which when intermixed makes the bricks less liable to crack. A troop of mares, sixteen to twenty in number, is then driven into the pit and kept constantly traveling round during a time varying from two to three hours according to the nature of the clay. At each step the hoofs of the animals sink to the bottom and the effect is readily imagined.

To the question, which naturally arises, why machinery is not used, the answer is that even though a large number of bricks could be made on one site without unduly increasing the cost of transportation, it would generally be a matter of no little difficulty to get the machinery in place ; and if this could be accomplished it would in most cases be next to impossible to secure the services of workmen capable of operating it efficiently.

The floors on which the raw bricks are placed for drying are prepared by smoothing the ground with spade and hand-scraper. A thin layer of sand or screened ashes is then applied to prevent adhesion between the bricks and the natural surface of the soil.

Before going further it might be well to state that the size of the bricks is usually about 11½ x 5½ x 2½ or 3 inches, which by the way is an economical and convenient size to employ in the construction of bridge piers and station houses. These diminsions have probably been introduced by Italians and Spaniards in later years, and are not those used by the old fashioned natives who still prefer the adobe bricks, which are 17 inches (a half vara) long, 8 inches wide, and 2 inches thick.

Two general methods are employed in moulding. One consists in placing a frame, having four or six divisions, on the floor and pressing the soft clay into the moulds, smoothing the top with the hands, then lifting the frame leaving the bricks behind. This is the quickest and cheapest way, but as the result is ordinarily defective bricks, particularly as regards shape, it is allowed only on inferior and unimportant works.

A better result is obtained by moulding on a table or bench. The moulds used are then either single or double, with or without bottom according to the desire of the brickmaker. Commonly they are dipped in water before the clay is pressed in, for the purpose of making the bricks slip smoothly from the sides when placed on the floor. This end is also attained by strewing the inside of the mould with fine sand. A good man working in this manner and with proper attendence can mould 2 000 brick per day. A fair average for each member of a gang of men doing all the work, including the preparation of the clay and loading of the kiln, would be about 500 per day.

The drying process is a matter of no little concern to the brickmaker, especially if "joint clay" or similar material has been used and a strong, hot wind is blowing. Various means are employed to prevent cracking. Sometimes a thin cover of sand is put on the top of the bricks shortly after moulding to protect them against the rays of the sun. In favorable weather they are left lying flat from one and a half to two days, when they are tilted and allowed to stand on edge about one day. They are then put in piles six to eight high, with wide openings between them, and when sufficiently dry to support considerable pressure they are placed on edge in long dikes about twelve high where they remain until ready for burning. The dikes are usually covered with corrugated iron sheets or with boards, and the sides exposed to the wind are protected either in a similar manner or by small bundles of weeds placed close together. The dikes stand on ridge like elevations covered with sand, and hence the bottom courses are less affected by the moisture of the ground.

In burning, or baking as it is sometimes called, the skillful brickmaker has a good opportunity to display his talent. His duties do not consist merely in loading the kiln and attending to the fires but embrace the construction of the flues.

The substructure of the kiln is ordinarily built as shown in Fig. 1, page 18. Sometimes the outer walls are carried up a foot or two above the flues, though this is usually done after the raw bricks have been put in place. The figures given are of course approximate as no fixed plan is adhered to. This work should not involve any considerable expense but it should nevertheless be sufficiently durable to withstand the action of more or less numerous fires. Second class bricks and mud constitute the building materials.

The fire openings, which extend from side to side through the kiln, are commonly bridged by projecting the bricks as shown.

In loading, the first three courses are put on edge, at right angles to and overhanging the flues so as to completely cover the openings. The fourth course is placed diagonally, and the following course is sometimes put on end, after which the rest all run parallel to the flues. This arrangement seems to be conducive to a favorable distribution of the heat throughout the kiln and to prevent undue deformation of the bricks. Its use can not be said to be general, as it is varied to suit different classes of material and, to some extent, the taste of the brickmaker. The height of the kiln when loam only is used is seldom more than ten feet above the flues, and

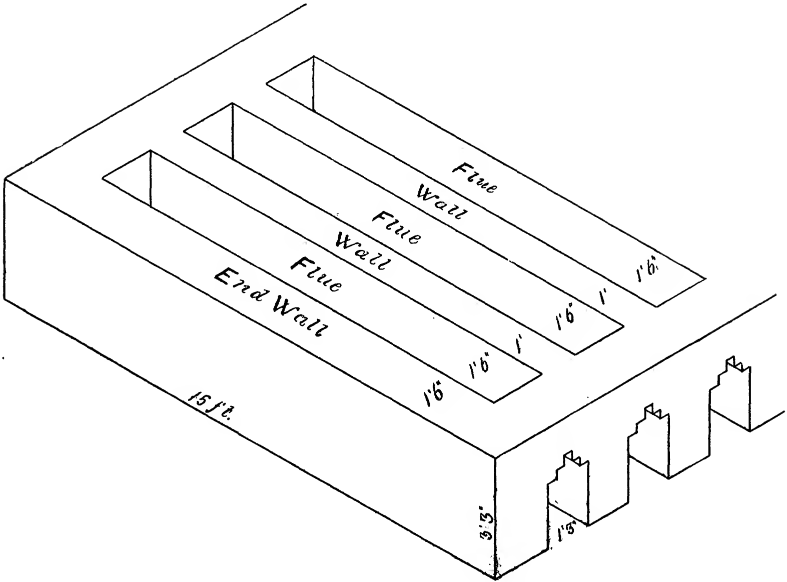

FIG. 1.

when the bricks are made of "joint clay" it is usually somewhat less. When loaded the kiln is covered on the sides and top with two layers of half burned, broken, or otherwise defective bricks laid flat. The sides are plastered with mud and the top is further covered with a layer of loose earth.

From 50 to 70 hours are required for burning, loam taking less time than the harder clays. During the first 12 or 16 hours a gentle fire is applied to drive all moisture out of the bricks, the temperature is then gradually raised to the full height. The effect of the fire is carefully watched and if indications of melting appear the fire in the corresponding flues is dampered and loose clay is thrown on the top of the kiln to send the draught through other parts. When the burning is complete the fire-holes are closed to exclude the atmosphere and prevent a rapid lowering of the temperature. The kiln is left in this condition for two days when openings are made in the cover to accelerate the cooling, and in the course of another day the fire-holes are opened and the cover removed.

The fuel ordinarily consists of branches and trunks of trees cut in lengths of four to five feet, of any thickness up to nine inches. A cord of wood will burn about 3 000 bricks.

In favorable weather a competent brickmaker can turn out 95 %

first class and 4 % second class material of all the bricks moulded; the remaining 1 % is lost in the various stages of the work.

In some parts of the immense pampas firewood is exceedingly scarce, and tall thistles and other large weeds are the only fuel available. In such cases short straw or other organic substance is often mixed in the clay when prepared for moulding. The bricks are then easily burned, some 18 hours fire being sufficient, but the material obtained is very porous, of little strength, and otherwise defective.

Perhaps it should be mentioned that in the aforesaid countries stone is not employed in the construction of bridges and culverts, except when found in the vicinity of the structure erected, and therefore the demand for bricks is correspondingly increased.

THE PONT-Y-PRYDD ARCH.

By W. D. Pence, Assistant in Applied Mechanics.

There are often associated with engineering structures of lesser magnitude many facts which, when known, prove to be of real interest and value. A notable instance of such is the Pont-y-Prydd masonry arch at Newbridge in south Wales. In presenting the following brief sketch, in which are collected from various sources the leading facts in regard to this arch, it is hoped to secure to its designer and builder some portion of the credit due him for this, his masterpiece.

In 1746 William Edwards contracted to bridge the river Taff at the site of the present Pont-y-Prydd, and completed a three-span arch, which, on account of its admirable execution, gained for its builder much praise. However, it seems that Edwards had failed to provide sufficient water way for the passage of extraordinary freshets, for one of the latter occurred only two and a half years after the completion of the bridge. The mountainous character of the upper Taff watershed and the union of several important branches above Newbridge caused a rapid concentration of water at the latter point during times of flood. In the particular freshet above referred to, much drift in the shape of trees, haystacks, etc., lodged against the piers of the newly completed structure and the waterway was soon entirely blocked. The water rose above the parapets, resulting in a short time in the total destruction of the arches and piers, the force of the current carrying the materials far down the river.

Being under bond to maintain a bridge for a period of seven years, Edwards soon resumed his work, and profiting by the experience gained in his previous attempt concluded to construct an arch of one span only. This was of course a much more difficult task, especially as the length of span determined upon was 140 feet, and the rise 35 feet, making the radius of the intrados 87 5 feet. This arch was completed with the exception of the parapet walls, when it failed upon removal of the centering by the sinking of the haunches and the consequent rising of the crown. However, this second misfortune did not prevent Edwards from making a third effort in which he was guided strictly by the experience gained in the previous failures. The span and rise were identical with those of the second arch, and in order to reduce the danger which had occasioned its destruction, Edwards devised the ingenious plan of placing over each haunch three cylindrical openings which materially reduced the weight at those points. It is also said that he filled the spaces between these openings with charcoal in order to still further reduce the load over the haunches.[*] The depth of the face stone is 2 feet 6 inches, while the interior portion of the arch has a thickness of only 18 inches. The roadway is 11 feet wide between parapet walls and is very steep, having a grade of apparently more than one in four. The width of the soffit at the crown is 14 feet 5 inches, increasing by six offsets on each face[†] to 15 feet 10 inches at the springing. The arch stones consist of thin slabs, the quarry beds of which were quite uniform and received very little dressing. The spandrel filling and parapet walls are of rough rubble. The masonry about the cylindrical opening was constructed with much care and the parapet walls were capped with a course of flat stones set edgewise. The masonry was laid in mortar made from Aberthaw lime, which has hydraulic properties.[‡]

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[*] Cresy's History of Civil Engineering, p. 426, London, 1861.

[†] Encyc. Brit. "Bridges" (Fleming Jenkin), Phila., 1891.

[‡] Obtained from a carboniferous limestone found near Cardiff, Wales, the composition of which before burning is (see "Notes on Building Construction," part 3, p. 155.)

Carbonate of Lime	86.2
Clay	11.2
Water	2.6
	100,0

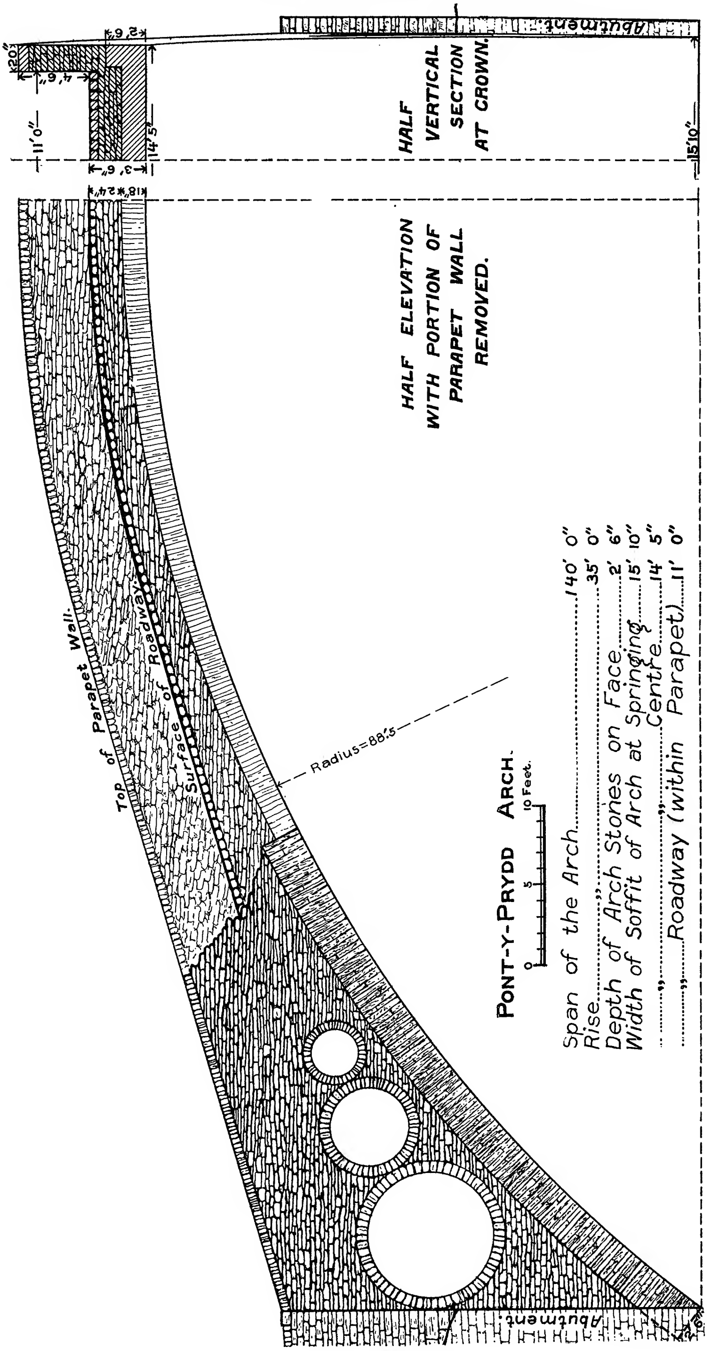
HALF VERTICAL SECTION AT CROWN.
HALF ELEVATION WITH PORTION OF PARAPET WALL REMOVED.
Top of Parapet Wall.
Surface of Roadway.
Radius=88'.5
Abutment.
Abutment.
2'0"
4'6"
11'0"
2'6"
14'5"
3'6"
18"×24"
15'10"
2'6"
PONT-Y-PRYDD ARCH.
0
5
10 Feet.
Span of the Arch..........140' 0"
Rise..........35' 0"
Depth of Arch Stones on Face..........2' 6"
Width of Soffit of Arch at Springing..........15' 10"
" " Centre..........14' 5"
" Roadway (within Parapet)..........11' 0"

FIG. 1.

This arch was successfully completed in 1755, nine years after the first bridge was begun and four years subsequent to the failure of the second. The final success of this hazardous undertaking gained for its promotor much praise locally; and it did not pass unnoticed among the earlier eminent civil engineers, for in 1809, when consulted upon a proposed bridge over the Thames similar in design to Peyronnet's celebrated Neuilly bridge, the great Rennie said:* "There is a bridge over the river Taff, in the county of Glamorgan, of upwards of 135 feet span, with a rise not exceeding 32 feet,† and what is more remarkable is, that the depth of the archstones is only 30 inches, so that in fact that bridge far exceeds in boldness of design that of Neuilly." Also, after referring to the well known excessive deflection in the French bridge, he adds: "No such change of shape took place in the bridge over the Taff (Pont-y-Prydd). The sinking after the centers were struck did not amount to one half of that of Neuilly, although the one was designed and built under the direction of the first engineer of France without regard to expense, whilst the other was designed and built by a country mason with parsimonious economy."*

The accompanying reproduction of a recent photograph of the Pont-y-Prydd shows its present condition. The excellent state of preservation of the rubble masonry after exposure for nearly a century and a half certainly indicates that much skill and excellent materials were used in the construction. The newer three-span bridge in the background has superseded the Pont-y-Prydd for general use, on account of the steepness of the roadway of the latter structure. In Fig. 1 are given some of the details of construction of the Pont-y-Prydd, based mainly upon dimensions given by Smith.‡ There is, however, a disagreement in regard to the diameters of the three openings over each haunch. Those used in Fig. 1 are 8, 5 and 3 feet respectively, which were carefully scaled from the only trustworthy drawing available.§ Cresy, an authority that seems to have been used by Smiles, gives the diameters as 9, 6 and 3 feet respectively. The latter set of dimensions, when tried upon the arch, do not give a satisfactory agreement with the photographic view. However, a series of very careful measurements made upon the photograph itself, using the depth of the face stones of the arch as a

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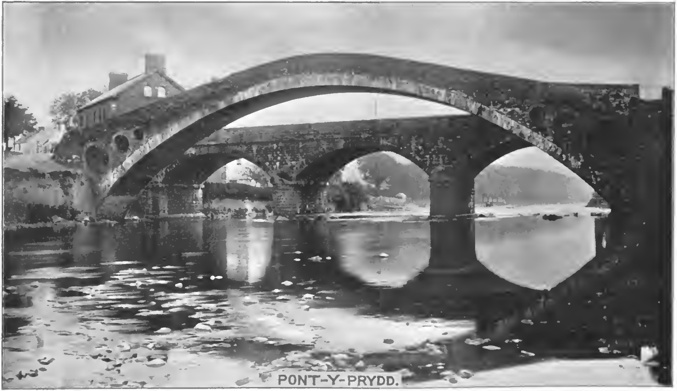
*Smiles "Lives of the Engineers," Vol. 2, p. 268, London, 1891.
†This dimension is in error. The rise is 35 feet.
‡Proc. Inst. C. E., Vol. V, p. 474, plates 38 and 39.
§Ibid., plate 38.



PONT-Y-PRYDD.

unit of comparison, has given diameters which agree in the two groups of openings at opposite ends with surprising closeness. The diameters thus determined are 9 feet, 5 feet 5 inches, and 3 feet 3 inches respectively. Taken in connection with other similar measurements as to the position of the holes with reference to the springing line and to each other, these dimensions have formed the basis of a careful investigation of the strains in the arch, as shown in Fig. 2.

In making this investigation, it is assumed that the loads acting upon the arch are vertical, which seems correct, inasmuch as the spandrel filling is rubble masonry laid in horizontal courses throughout. If the charcoal filling were used, a saving of over 100 pounds per cubic foot would be effected, provided the charcoal be dry, the difference decreasing of course with the increase of moisture absorbed. However, as this filling, if used at all, would probably have been confined to the small space above and between, rather than beneath, the masonry barrels over the haunches, the effect upon the loads in that portion of the structure would be relatively small. It is further assumed that the surplus in the depth of the face stones over that of the interior arch stones is sufficient to bear the weight of the parapet walls in such a manner that the entire arch acts as a unit. In Fig. 2, a longitudinal slice one foot thick is assumed, the upper limit being the surface of the roadway. Subdivisions of the intrados of five degrees each are taken, and the sectional areas are computed, and the centers of gravity determined on the assumption of uniform density of masonry in both arch and filling. The crown thrust was then computed by taking moments of all the loads acting between the crown and each joint, successively, for assumed positions of the center of pressure in both the crown joint and in the joint considered, the center of moments being in the latter. Two assumptions were made for each joint; viz., (the usual one) that the center of pressure in the crown joint is at the *upper* limit, and in the joint considered at the *lower* limit, of the middle third; and, vice versa, that the center of pressure in the crown joint is at the *lower* limit, and in the joint considered at the *upper* limit, of the middle third.

Under each of the above assumptions the crown thrust was found to be a maximum when considering the entire arch, which locates the "joint of rupture" at the springing line. These crown thrusts, amounting to the weight of 314.0 and 326.4 cubic feet of masonry, are used in Fig. 2, in the construction of the two lines of resistance, shown in full and dotted lines, respectively.

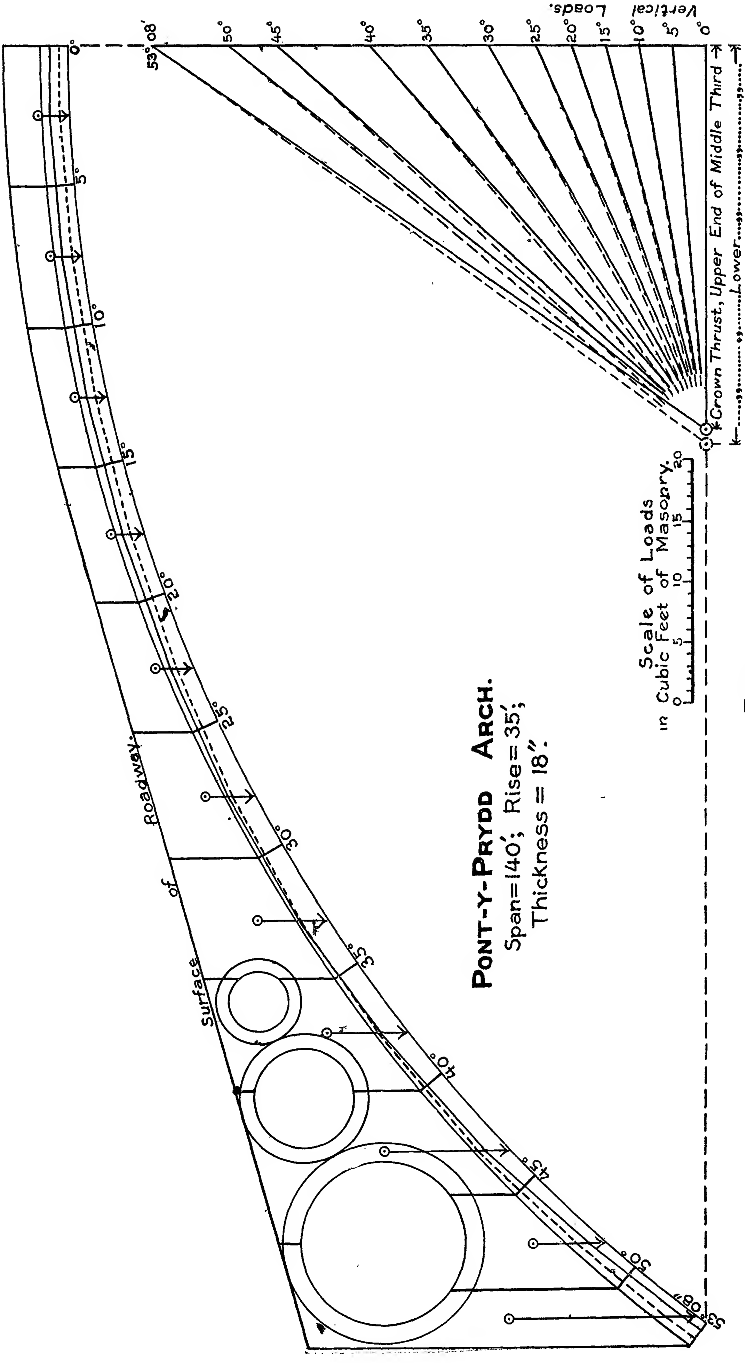

Pont-y-Prydd Arch.
Span=140'; Rise=35';
Thickness = 18".
Scale of Loads
in Cubic Feet of Masonry.
0
5
10
15
20
Vertical Loads.
0°
5°
10°
15°
20°
25°
30°
35°
40°
45°
50°
53° 08'
Crown Thrust, Upper End of Middle Third
Lower
Surface of Roadway.

FIG. 2.

Table I, below, gives the maximum pressure in pounds per square inch for each joint under each of the above assumptions. The weight per cubic foot of rubble masonry is taken at 150 pounds,* and it is assumed that no tensile resistance is given by the mortar in the joints of the arch.

TABLE I.

MAXIMUM PRESSURES AT GIVEN POINTS IN THE PONT-Y-PRYDD ARCH.

POSITION OF JOINT.	PRESSURE IN POUNDS PER SQUARE INCH.	
	1ST ASSUMPTION.	2ND ASSUMPTION.
0° (crown)	436	452
5°	437	454
10°	442	458
15°	450	428
20°	501	318
25°	722	370
30°	1 000	615
35°	929	602
40°	580	1 025
45°	382	1 011
50°	640	710
53°08′ (springing)	768	778

No records as to the probable compressive strength of the stone used in the arch are available, but its condition after so long an exposure indicates a firm texture. With a resistance of 5 000 pounds (2½ tons) per square inch, the factor against crushing would nominally be about 5, but would actually be greater than this on account of the favorable slab-like form of the arch stones. However the factor against rotation is only 1.5, inasmuch as the maximum deviation of the line of resistance from the center line of the arch stones is 6 inches.

It may be stated that a similar determination of the strains was made with the position and size of holes as in Fig. 1, the results of which did not differ greatly from those given above. An investigation of the arch, omitting the holes entirely, would also be of interest; but with the uncertainty in regard to the amount of parapet wall completed at the time of the failure of the second arch, the results would not be conclusive as to the cause of the failure.

A description of this remarkable structure is incomplete without a passing notice of its designer and builder. William Edwards, who was born in 1719, was the youngest son of a Glamorganshire farmer. His natural ability, shown first in the construction of dry stone farm fences, developed so rapidly that when scarcely of age he was employed to build an iron forge at Cardiff, a contract which was satisfactorily completed. As occasion offered, he visited and studied with much profit to himself, the extensive ruins of Caerphilly Castle near by, and by making frequent application of the principles of construction there observed, his progress was deservedly rapid. It was there that he gained his knowledge of the principles of the arch, which he first applied in building a mill near his native place.

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* "Notes on Building Construction," Vol. 3, p. 69, gives 136 lbs. per cu. ft. as the weight of a limestone much used for older structures in south Wales.

The contract to bridge the Taff in 1746—when Edwards was but 27 years of age,—the destruction of the first, the fall of the second, and the successful completion of the third arch, have already been described. Edwards's inexperience in bridge design will excuse the first failure, if indeed the extraordinary character of the flood was not the sole cause. The second failure may justly be attributed to the premature striking of the centers, a natural error in view of the danger in keeping the channel of the river obstructed for a great length of time; also it was quite natural to complete the spandrel filling and parapet walls over the center of the arch last, which would also contribute to the cause of failure. All these dangers were foreseen and provided for in the third effort.

The Pont-y-Prydd has been referred to as "a fine example of that ignorance which often passes for boldness."* While it is doubtless true that Edwards was not aware of the fact that the span chosen for his bridge exceeded that of any masonry arch then existing, and however appropriate the above accusation may have been against the earlier and unsuccessful arch, the existing structure certainly does not deserve it. And, too, it would seem that common justice to the zeal and energy which won for him many successes in bridge building, of which the Pont-y-Prydd was first and most noteworthy, should set aside as inappropriate and outgrown the title of "uneducated country mason," so often used in this connection.

It is hoped that there may in future be less obscurity about this remarkable arch, and that in searching the records of men noted for their zeal and inflexibility of purpose, we may not fail to find the name of

William Edwards

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*Trautwine's Engineers' Pocketbook, p. 696.

TRANSVERSE STRENGTH OF BRICK MASONRY.

By M. A. Earl, '93, and A. B. Loomis, '93, Civil Engineering Course.

INTRODUCTION.

In spite of the fact that brick masonry is very generally used in engineering structures, comparatively few experiments have been made to determine its transverse strength. In fact, this strength has not usually been counted on in designing, as so little definite knowledge was to be had concerning it. For instance, when an opening for a door or window is cut in a brick wall, it is known that the masonry immediately above is subjected to transverse strains. Transverse strains also come in walls subjected to water or wind pressure and in tall chimneys. Brick arches, which theory shows would fall, considered simply as arches, do not fall, and it is undoubtedly the transverse strength of the masonry which prevents failure.

"Much surprise was expressed, at the time, at the comparatively enormous strength of some narrow strips of brick arches, 15 feet span, and only 4½ inches thick, tested by Mr. Fairbairn; but there were no just grounds for surprise."* The surprising strength of the arch was due to the transverse strength of the masonry.

The supports of masonry over doors and windows are sometimes considered as supporting the entire weight of the masonry above them. This involves the idea of fluid brick and is fallacious. When a wall is "green" it will have to be supported above openings, but if built up slowly the supports may be removed in many cases as soon as the lower courses have set. Knowing what modulus of rupture can be relied upon, the computation is simple.

PART I.

EXPERIMENTS BY OTHERS.

Investigation shows the literature on this subject to be exceedingly limited. The experiments made have been few and poorly performed. The following references bear more or less directly

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**Engineering*, Vol. 14, p. 73.

upon the subject: *Engineer and Architect's Journal*, vol. 1, pp. 30, 45, 102, 135, (1837); vol. 11, p. 294, (1848); vol. 14, p. 510, (1851). *Engineering*, vol. 14, p. 1, (1872); *Indian Engineering*, Jan. 9, 1892, or *Railroad Gazette*, Feb. 26, 1892.

We will give a brief account of some of the most important experiments mentioned in these articles.

BRICK AND CEMENT BEAMS.*

Three experimental brick beams were built and broken by weights applied at the center. The beams were 10 feet long, 18 inches wide, and 12 inches deep. No. 1 was built of bricks with neat cement joints. No. 2 was the same, but had five pieces of hoop iron in the joints, one at center joint and two at each of the other joints. No. 3 was built with hoop iron and lime. One object in the experiment was to ascertain the additional strength given by the hoop iron. No. 1 broke with a load of 298 lbs. $S=$modulus of rupture $=13.8$ lbs. per sq. in. The bricks themselves were broken.

No. 2 broke with a load of 4 779 lbs. The hoop iron apparently increased the strength 16 times.

THE "NINE ELMS" BRICK BEAM.

This was a brick wall 24 feet 6 inches long. Distance btween supports 21 feet 4 inches. It supported about 11 tons for two years. There were seventeen pieces of hoop iron laid longitudinally in the beam. The mortar used was 1 part Roman cement to 1 part sand. The total breaking load was 68 326 lbs. The modulus of rupture given by *Eng'r and Arch't's Jour.* is "$S=\frac{lw}{4ad^2}=641$" lbs. per sq. in. *Engineering*, vol. 14, p. 45, gives $S=290$ lbs. per sq. in. We believe the latter to be the correct value. "There is no doubt that the iron supported by far the greater part of the weight." Some claimed that the hoop iron added practically nothing to the strength of the beam.

"Subsequent experiments have shown that the truth was to be found between these extremes, and that the hoop iron contributed just as much, and no more, additional strength as theory would indicate. In the case of the "Nine Elms" beam this would be a comparatively small amount, since the hoop iron was of no great section and the relative elasticities of the materials are such that in no case

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**Eng'r. and Arch't's Jour.*, vol. 1, p. 30.

could a large strain be imposed upon the iron in a compound beam until after the fracture of the brick work had occurred."

EXHIBITION BEAM OF 1851.†

A brick beam, very similar to the "Nine Elms" beam, was constructed of hollow and Portland cement mortar, 1 to 1.

Dimensions: Span, 21 feet 4 inches; depth, 4 feet 6 inches; width, 1 foot 6 inches. The beam cracked when the concentrated, load reached 41 600 lbs., and broke with a load of 62 800 lbs.

There were fourteen strips of hoop iron, 1½ inches×1-16 inch, reaching through the beam. Seven strips were broken and seven were drawn out. Modulus of rupture = 350 lbs. per sq. in. The strength was due largely to the iron, and the experiment gives no idea of what the beam might otherwise have held.

EXPERIMENTS ON CEMENT.‡

The following experiments, a summary of which is given on page 32, were made at Great Scotland Yard:

"1. Seventeen stock bricks were cemented together with Roman cement and projected from the face of a wall. They broke down with 7 ℔s. placed on the end.

"2. Eleven stock bricks, cemented together with 1 part sand and 1 part Roman cement, broke down with 7 ℔s. placed on the end.

"3. Thirty-eight bricks, cemented with neat Portland cement broke down with 14 ℔s. placed on the end.

"4. Thirty bricks, cemented with 1 part Portland cement and 1 part sand, broke down with 15 ℔s. at the end.

"6. Twenty-five bricks, with 1 part Portland cement and 4 parts sand, broke down with 56 ℔s. at the end.

"7. Twenty-six bricks, with 1 part Portland cement and 5 parts sand, broke down with 74 ℔s.

"8. Sixteen bricks, with 1 part Portland cement and 1 part sand, supported at both ends, broke down with 1500 ℔s. at the center."

The unreliability of these experiments is shown by a comparison of 4 and 9 with 5, 6, or 7; and by comparing 6 and 7 in the following table:

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†*Eng'r and Arch't's Jour.*, vol. 14, p. 510.

‡*Eng'r and Arch't's Jour.*, Vol. 14, p. 510.

TABLE I. TESTS OF CANTILEVER BRICK-BEAMS.*

NO. OF BEAM	NO. OF BRICKS.	CHARACTER OF MORTAR.	BREAKING LOAD.	MODULOUS OF RUPTURE.
1	17	Roman Cement, Neat.	7 lbs. at end.	13 lbs. per sq. in.
2	11	“ “ 1 to 1.	7 “ “ “	9 “ “ “
3	38	Portland “ Neat.	14 “ “ “	330 “ “ “
4	30	“ “ 1 to 1.	15 “ “ “	210 “ “ “
5	22	“ “ 1 to 2.	168 “ “ “	310 “ “ “
6	25	“ “ 1 to 4.	56 “ “ “	210 “ “ “
7	26	“ “ 1 to 5.	74 “ “ “	247 “ “ “
8	16	“ “ 1 to 1.	3000 “ at center.	234 “ “ “

Eng'r and Arch't's Jour., Vol. II.

INDIAN BEAMS‡

Thirty beams were tested. All were about 30 inches square and were broken with a span of 10 feet. They were made in Dec., 1888. One half were tested in Feb., 1889 and half in Nov., 1889. The joints were $\frac{1}{4}$ inch thick. The article does not give the character of the mortar.

Three kinds of brick were used: (A) Almost wholly of bats or new half bricks with three whole bricks in the header course every 30 inches, to insure a slight bond. (B) Whole bricks once used, called demolition bricks. (C) New whole bricks.

The beams were loaded with steel rails, laid on sleepers 4 inches by 8 inches and 16 feet long placed across the beam near the center.

In computing S the equivalent uniform load was used and the weight of the beam was included. The formula for S is

$$S = \frac{3Wl}{2bh^2}$$

TABLE II. STRENGTH OF INDIAN BRICK BEAMS.

PERIOD OF SETTING.	MODULOUS OF RUPTURE, IN POUNDS PER SQUARE INCH.			
		CLASS A.	CLASS B.	CLASS C.
		Nos. 1 to 7 Inclusive.	Nos. 1 to 5 Inclusive.	Nos. 1 to 5 □ Inclusive.
70 to 76 Days.	Maximum	87.78	108.86	112.67
	Minimum.	75.74	98.86	94.00
	Average	79.78	102.86	103.34
		Nos. 8 to 10.	Nos. 6 to 10.	Nos. 6 to 10.
260 to 270 Days.	Maximum	94.00	145.51	150.00
	Minimum.	78.45	112.19	109.56
	Average	87.78	126.19	133.20

‡*Indian Engineering*, Jan. 9, 1892.

PART II.

EXPERIMENTS BY THE AUTHORS.

The following experiments were performed by the writers in the testing laboratory of the University of Illinois, and represent as nearly as may be the conditions of actual practice.

MORTAR USED.

The mortar for the first beam and first pier was made thus: One part by weight of Louisville cement to two parts of sand, *i. e.*, 1 volume of cement to 1.35 volumes of sand. The mortar for the remainder of the beams consisted of 1 volume of cement to 2 volumes of sand.

In making these experiments the mortar was never frozen although nearly reaching that temperature some of the time.

The mortar was laid within 15 or 20 minutes after water was added to the cement.

The bricks were moistened before laying and the mortar made of about the same consistency as ordinary lime mortar.

TESTS OF CEMENT.

Fineness. 93.3 % passed sieve No. 50 (2 500 meshes per square inch); 86.3 % passed sieve No. 80; 83.8 % passed sieve No. 100. This was on the basis of 100 % of original cement placed on each sieve.

Tensile Strength. The tensile strength was determined by breaking American standard briquettes on a Riehle testing machine.

TABLE III. TENSILE STRENGTH.

POUNDS PER SQUARE INCH.

AGE—7 DAYS.			AGE—24 DAYS.		
NEAT.	1 TO 2 BY WEIGHT.	1 TO 2 BY VOLUME.	NEAT.	1 TO 2 BY WEIGHT.	1 TO 2 BY VOLUME.
79	28	19	118	38	28
61	24	16	116	35	31
74	26	19	114	35	21
81	26	14	130	39	35
74	26	17	125	40	34
74	26	17	121	37	30

TEST OF SAND.

Fineness. The sand used had all passed a No. 12 sieve (144 meshes per square inch), 98.3 % passed a No. 20 sieve; 78.3 % passed a No. 30 sieve. Coarser sand was not available.

BRICKS USED.

The bricks measured 7 inches by $3\frac{1}{2}$ to $3\frac{3}{4}$ inches by $2\frac{1}{8}$ to $2\frac{1}{4}$ inches. They were ordinary sand-molded building brick. Average weight=4.45 lbs.

Transverse Strength. Ten bricks were broken as simple beams and the following values of the modulus of rupture obtained: 606, 1 348, 606, 808, 1 313, 1 666, 1 010, 1 010, 1 010 and 960 lbs. per square inch. Mean value, 1 034 lbs. per square inch.

DESCRIPTION OF THE BEAMS.

To obtain values of the modulus of rupture of brick masonry eight beams and two piers were built and broken. The beams were broken as simple beams with concentrated load at center. The piers were first broken as cantilever beams and afterwards as simple beams.

Beam No. 1—Distance between supports=43.75 ins. Age when broken = 28 days.

The beam failed at the center, about as much from lack of cohesion in the mortar as from mortar separating from the brick. This statement will apply to all the beams. One brick was broken by tension.

Since the weight of the beam (350 lbs) acts as a uniform load, its effect is the same as 175 lbs. at the center. Hence the breaking load $=1652+175=1827$ lbs. Therefore,

$$\text{modulus of rupture} = \frac{6M}{d^2b} = \frac{6\times1827\times43.5}{4\times7.75\times217.56}$$

Pier No. 1. Pulled over as a cantilever beam. Age=28 days. Failed by mortar separating from brick. Weight of pier=295 lbs. Force to break=205 lbs. applied 42.5 ins. from joint of rupture. Maximum tension per sq. in.

$$=\frac{Ml}{2\,I}-\frac{W}{S}=\frac{205\times42.5\times11\frac{1}{2}}{2\times\frac{1}{12}\times7\frac{1}{2}\times(11\frac{1}{2})^3}-\frac{295}{7\frac{1}{2}\times11\frac{1}{2}}=49.3\text{ lbs.}$$

Beam No. 2 consisted of pier No. 1 broken as a simple beam. Length between supports, 43 ins.; depth=$7\frac{1}{2}$ ins.; width=$11\frac{1}{2}$ ins.

Weight 295 pounds. Load to break: concentrated, 160 lbs.; weight of beam considered at center, 148 lbs.

$$S=\frac{6M}{b\,d^2}=\frac{6\times 308\times 43}{4\times 11\tfrac{1}{2}\times (7\tfrac{1}{2})^2}=30.7 \text{ lbs. per sq. in.}$$

In this case, and also in the pier, the unit stress is necessarily small since there is no interlocking action of the brick as in an ordinary beam.

Beam No. 3. Built essentially the same as No. 1. Age=56 days. Length=4 feet; depth=4¾ inches; width=7¾ inches; distance between supports=43 inches. Weight=350 pounds. Concentrated load=3 273 pounds. The beam did not fail, thus showing a strength of at least $=\frac{6M}{b\,d^2}=\frac{6\times 3\,448\times 43}{4\times (04\tfrac{3}{4})^2\times 7\tfrac{3}{4}}=117.2$ lbs. per sq. in.

It was then turned on the flat side and broken with a concentrated load of 1 507 lbs. $S=\frac{6\times 1\,682\times 43}{4\times(14\tfrac{3}{4})\times(7\tfrac{3}{4})^2}=122.4$ lbs. per sq. in.

Beam No. 4. Age=49 days; depth and width consisted of the same number of courses as No. 1; depth=15 inches; distance between centers=122 inches; width=7¾ inches; weight=1 000 lbs.

Five bricks broke when the beam failed. When this beam was broken deflections were taken at the center by means of a level and rod. These are given in Table IV.

TABLE IV. Deflections.

DEFLECTION IN INCHES.	DIFFERENCES.	WEIGHTS APPLIED.	
0.00	0.00		0.00
0.05	0.05	WT. OF BEAM	
0.06	0.01	"	+ 200 lbs.
0.07	0.01	"	+ 400
0.11	0.04	"	+ 600
0.17	0.06	"	+ 800
0.23	0.06	"	+1000
Beam Broke		"	+1224

At the vertical joints the mortar separated from brick; at the horizontal joints the mortar failed in cohesion. $S=\frac{6\,M}{b\,d^2}=\frac{6\times 1\,724\times 122}{4\times(15)^2\times 7\tfrac{3}{4}}$ $=181$ lbs. per sq. in.

Beam No. 5. Depth=15 inches; width=7¾ inches; distance between supports=55 inches; weight=500 pounds; age=61 days; breaking load applied at center=1 678 lbs. $S=\frac{6\,M}{b\,d^2}=\frac{6\times 1\,928\times 55}{4\times 7\tfrac{3}{4}\times(15)^2}$ $=91.2$ lbs. per sq. in.

Beam No. 6. Width and depth same as No. 5; distance between supports=55 inches; age=62 days; load to break =2 070 lbs; weight of beam=500 lbs. $S=\frac{6\times 2\,320\times 55}{4\times 7\frac{3}{4}\times (15)^2}$=115.5 lbs. per sq. in.

Beam No. 7. Age=62 days; distance between supports=44 inches; depth=$7\frac{1}{4}$ inches; width=$3\frac{1}{2}$ inches; center load=378 lbs; weight of beam=80 lbs. Beam was not broken.

$S=$ at least $\frac{6\,M}{bd^2}=$ at least $\frac{6\times 418\times 44}{4\times (7\frac{1}{4})^2\times 3\frac{1}{2}}$=at least 153.8 lbs. per sq. in.

Beam No. 8. Two courses high and two courses wide. Depth $=4\frac{1}{2}$ inches; width$=7\frac{1}{2}$ inches; distance between supports=45 inches; weight of beam=125 pounds; breaking load=200 pounds; age=62 days; $S=\frac{6\times 45+262}{4\times 7\frac{1}{2}\times (4\frac{1}{2})^2}$=112.7 lbs. per sq. in.

Beam No. 9. Same section as beam No. 2. Built as a pier. Distance between supports=37 inches; width=$7\frac{1}{2}$ inches; depth=$11\frac{1}{2}$ inches; age=62 days; weight of beam=270 lbs.; center load=970 lbs. $S=\frac{6\times 970\times 37}{4\times 7\frac{1}{2}\times (11\frac{1}{2})^2}$= 54.3 lbs. per square inch.

Beam No. 10. Age=35 days. Distance between supports=6 feet; depth=14 inches; width=8 inches; center load=1 000 lbs.; weight of beam=670 lbs. $S=\frac{6\times 1\,435\times 72}{4\times 5\times 196}$=99 lbs. per square inch.

TABLE V. SUMMARY OF RESULTS.

NO. OF BEAM..	AGE IN DAYS..	PROBABLE TENSILE STRENGTH OF MORTAR AT THAT AGE IN LBS. PER SQ. IN.	MODULUS OF RUPTURE OF BEAM IN LBS. PER SQ. IN.	REMARKS.
3	56	50	117.2.......122.4	Broken Sideways.
4	49	48	181.	Mortar 1 to 2 by Volume.
5	61	52	91.2	"
6	62	52	115.5	"
7	62	52	153.8	"
8	62	52	112.7	"
10	35	45	99.0	"
9	62	52	54.3	" Built as a Pier.
2	28	40	30.7	Mortar 1 to 2 by Weight.
1	28	40	71 1	" " " "
PIER	28	40	49.3	" " " "

CONCLUSION.

The table shows, roughly, that the beams built as regular masonry have a modulus of rupture of about twice the tensile strength of the mortar used. With the best construction we believe it may be even three times the tensile strength of the mortar, as shown by beams No. 4 and 7. When built as piers, with no interlocking action, the modulus of rupture is about the same as the tensile strength of the mortar used.

The experiments on deflections with beam No. 4 while not enough to draw any certain conclusion from, would seem to show that brick masonry is elastic and that up to a certain point the deformation is proportional to the stress applied.

The result of experiments in the past, while showing a certain transverse strength, have not been definite or uniform enough to furnish reliable conclusions. We believe we have shown that this strength can be counted upon in designing.

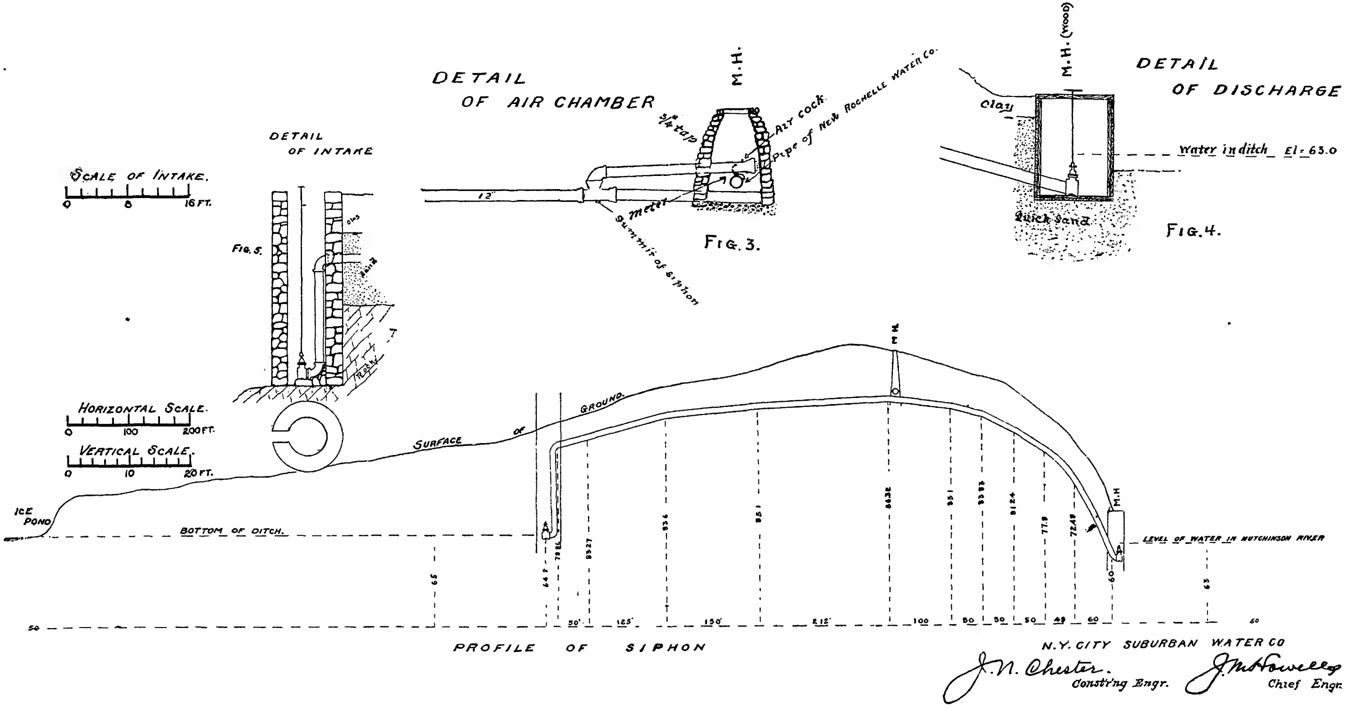
While the nature of this subject does not permit of its being carried in an experimental way as far as might be desired, without considerable expense, we would suggest that much can yet be done towards finding the strength actually obtained in brick buildings. The tearing down or failure of such buildings affords an excellent opportunity for this and it is to be hoped that experiment and observation will be turned in this direction.

A BIG SIPHON.

By J. N. Chester, '91. Constructing Engineer, New York City Suburban Water Co.

The writer recently had charge of the construction of a large siphon, which on account of its unusual lift and great length may be worthy of description. This siphon has a length of 925 feet, and its maximum lift of 23 feet is made with a head, or difference in level of the water at the intake and outlet, of only 2 feet. So far as ascertained these conditions are far more severe than those of any siphon on record.

Mount Vernon, a city of 18,000 inhabitants, on Long Island Sound, 14 miles northeast of the Grand Central Depot, New York City, is furnished water by the New York City Suburban Water Company. Up to 1892 the source of supply was the Pelhamville res-


DETAIL OF AIR CHAMBER
M.H.
3/4" tap
Air cock
Pipe of New Rochelle Water Co.
meter
Summit of siphon
12"
FIG. 3.
DETAIL OF DISCHARGE
M.H. (WOOD)
clay
Water in ditch El. 63.0
Quick Sand
FIG. 4.
DETAIL OF INTAKE
Fig. 5.
clay
sand
Rock
SCALE OF INTAKE.
0
8
16 FT.
HORIZONTAL SCALE.
0
100
200 FT.
VERTICAL SCALE.
0
10
20 FT.
ICE POND
BOTTOM OF DITCH.
SURFACE OF GROUND.
M.H.
M.H
LEVEL OF WATER IN HUTCHINSON RIVER
50
65
64.9
78.85
83.27
83.6
85.1
86.32
85.1
83.83
81.24
77.8
72.49
60
63
50'
125'
150'
212'
100
50
50
50
49
60
60
PROFILE OF SIPHON
N.Y. CITY SUBURBAN WATER CO
J. N. Chester.
Constr'ng Engr.
J. McHowells
Chief Engr.

ervoir on Hutchinson river at the point where it is crossed by the New York, New Haven and Hartford Railroad. The watershed of the Hutchinson river above this point was 8 miles long and contained 3 000 acres, but the reservoir of the New Rochelle Water Company six miles up the river cut off 1 000 acres of this. The further construction of another reservoir three miles up by the New Rochelle Water Company in 1891, through a prior right, reduced the available water shed to 1 000 acres. About May 1, 1892, the water ceased to flow over the spillway of the Pelhamville reservoir and by August 1 the flow into the reservoir stopped. As the reservoir holds only 17,-000 000 gallons and as the daily consumption by Mount Vernon was about 1 250 000 gallons, the problem of securing water until December 1, when the flow of the stream would probably become equal to this daily consumption, was foreseen to be a grave one.

In May, 1892, Mr. J. N. Howells of Chicago, made an examination for an additional supply and settled on the Tom Paine brook as the source of supply, and upon the Mahlsled ice ponds and vicinity as reservoir sites. At this point, which is just below the lower reservoir of the New Rochelle Water Company, the brook is within one half mile of the Hutchinson river. In order to conduct the water to the Pelhamville reservoir, Mr. Howells decided to construct a siphon over the divide between the brook and the river and thence allow it to flow either by pipe or through the river channel one and three fourths miles to the reservoir. The construction of the new reservoirs required time, but the emergency necessitated the immediate construction of the siphon and channel, and as the result of our effort the water from the ice pond was flowing through within four weeks from the commencement of the work.

The siphon is 925 feet long and is made of 12-inch pipe. The profile, manholes, valves, air chamber and priming supply at the summit are shown in the cuts. During priming the valves at the inlet and outlet are closed and the supply taken through a meter from the main of the New Rochelle Water Company till the water spurts from the air cock. The latter is then closed and the valves opened. The discharge has been found by weir measurements to be approximately the same as that in a pipe of the same length and size under a head equal to the difference in level between the water at the two ends, as long as the accumulated air is contained within the air chamber. With air in the siphon the flow of course diminishes and finally stops altogether. Much of this air is given off from the water, but while the greatest care in calking the joints possible under the circumstance of construction was taken, some of it enters through leaky joints, as the pipe is very poor, so poor that it would certainly have been rejected if time had allowed other pipe to be secured. It is impossible to say what proportion is received from leaky joints. As constructed it requires 5.5 hours to charge the empty pipe and 1.25 hours to run in 125 cubic feet. If charged every 6 hours, but 15 minutes is required, and one man can keep it running 23 hours out of 24 and have plenty of time to sleep. The accumulation of air is about twice as rapid with a lift of 23 feet as it is at 17 feet. With a lift of 20 feet, 125 cubic feet of air accumulates in 12 hours.

Among the things that this experience in siphon building shows are that a rapid method of filling should be provided and that the air chamber should be proportioned to the lift. When water is relieved of the atmospheric pressure, it gives up the air held by it much more rapidly as the pressure increases. Our air chamber will be enlarged to contain 125 cubic feet. This will be constructed as shown in Fig. 1 and 2.

To pump the water over the divide or to lay the pipe to grade as a conduit would have been much more expensive and would have taken more time for construction. To lay the pipe to grade would have required a cut of 37 feet at the highest point, 25 feet of which would be rock. Considering the conditions, the results obtained from the siphon have been very gratifying.

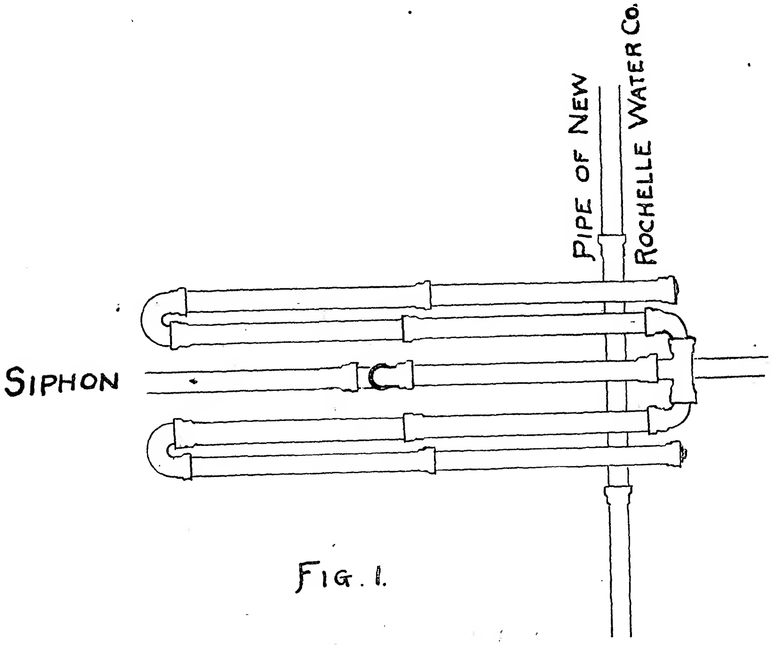


FIG. 1.

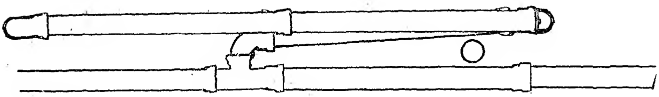


FIG. 2.

STRETCHING CROSS-HAIRS.

ARCHITECTURAL SHOP.

GAS ENGINE VALVE SETTING.

W. H. Cornell, '93, and B. V. Swenson, '93, School of Mech. Eng.

The engine in question is the one in the electrical laboratory of the University, and was put in for the purpose of running dynamos. It is known as the "Cycle," and was manufactured by Henry Warden of Philadelphia, under Atkinson's patents. The engine was set up, but for some reason, refused to run.

Considerable time was spent on it, but with little success, and no one seemed to be able to discover what the trouble was.

In January, 1893, we determined to make an investigation, and if possible to locate the difficulty. When we began our work, the engine would run, but on applying the friction brake, it was found that all, or nearly all, of the power was used in overcoming the friction of the engine.

At that time the chief difficulty seemed to be in securing the ignition of the charges. Ignition is secured bv compressing the charge, and thus forcing the gas into a small tube, closed at one end, and heated to incandescence by the flame of a bunsen burner. The difficulty was caused by the suction of the engine, which reduced the bunsen flame to such an extent that the temperature of the tube was too low to ignite the charge. The air holes of the burner were enlarged, giving a much better fiame, and a gas bag inserted in the burner supply pipe. After this we had no further trouble with it.

We found the piping in a very defective condition, the jacket water being admitted from the top of the cylinder instead of from below, making it impossible to keep the cylinder cool, and the gas pipes so small and so full of elbows, as to seriously impede the flow. The piping was changed, and a larger gas pipe inserted. It did not appear, however, that the efficiency of the engine was materially increased, and an investigation of the valve setting was begun.

An indicator was attached, and cards taken. Card No. 1, Plate 1, was taken, with the engine in its original condition. As it was taken with a 100 pound spring the faults of the valve-setting are not very apparent. We then determined to use a 20 pound spring, and turn the engine over by hand. Card No. 2 was the result. To understand the cards, it is necessary to understand the action of the engine. By referring to Plate 2 it will be seen that the valves are of the poppet

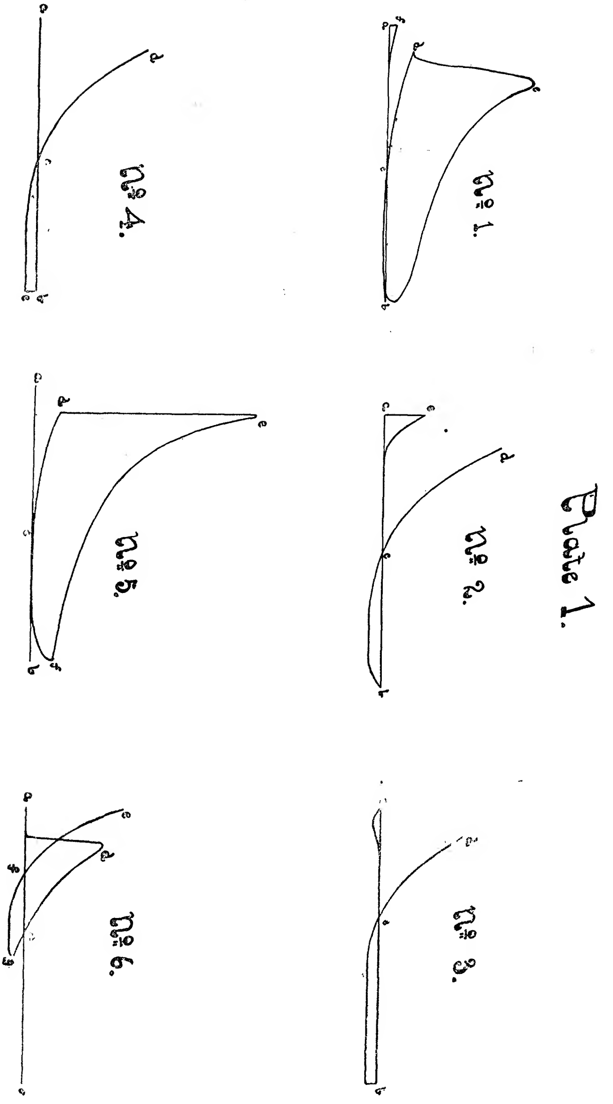


Plate 1.

pattern, opened by cams, and brought to their seats by powerful springs. The piston makes four strokes, two long and two short, during each revolution of the fly wheel. Referring to Card No. 2, and supposing that the piston is at *b*, the events of the strokes are as follows; The piston moves from *b* to *a*, making the exhaust stroke.

It then moves forward to *c*, making the admission stroke, then back to *d*, making the compression stroke, at the end of which explosion takes place, the piston moving from *d* to *b*, making the working stroke, and completing the cycle.

The action of the valves should be as follows:

The exhaust valve should open at *b*, and remain open until the piston returns to *a*. It should now close, and the admission valve open, remaining open until the piston reaches *c*. It should then close, and remain so until the piston is again at *a*. That this is not the case is shown very clearly by card No. 2. It is plain that the exhaust valve closes and opens too early in the stroke, losing considerable power by introducing compression at the end. This causes a considerable amount of the products of the combustion to remain in the cylinder, diluting the succeeding charge, and thus reducing the effect of the explosion. By referring to Plate 2, it will be seen that both cams are on the same shaft. The shaft is driven by a worm gear connected with the main shaft in such a manner that the cams make the same number of revolutions as the fly-wheel. By disconnecting the worm gear, the cams may be rotated back or forward while the fly-wheel is at rest, thus changing the events of the stroke.

Card No. 2., shows by the line *a e*, that the admission valve opens at the proper time, but whether it closes at the proper time can not be told because admission takes place on the atmosphere line. How it is told will be shown later. It remains therefore to cause the exhaust valve to work properly. The gear was disconnected, and the cams rotated back one cog of the worm wheel. Card No. 3, shows the effect. It is seen that the exhaust valve works perfectly, while the loop, at *a* shows that the admission valve opens late, and as the line *a c* is shorter than *a c* in card No. 2 it shows that it also closes late, thus allowing a part of the charge to be driven back into the airpipe, reducing the effect of the explosion more than the retention of the waste gases did, and leaving the engine in worse condition than before. To remedy this, it was necessary to reduce the angle between the cams. The cams are firmly fastened to the shaft by means of set screws, and are not supposed to be adjustable. The screws were removed, the cam was turned forward, a new seat for the set screw drilled, and card No. 4 was taken. This shows perfect action of the valves, as by card No. 6, in which the line traced during the admission stroke lies below the atmosphere line, the point at which the

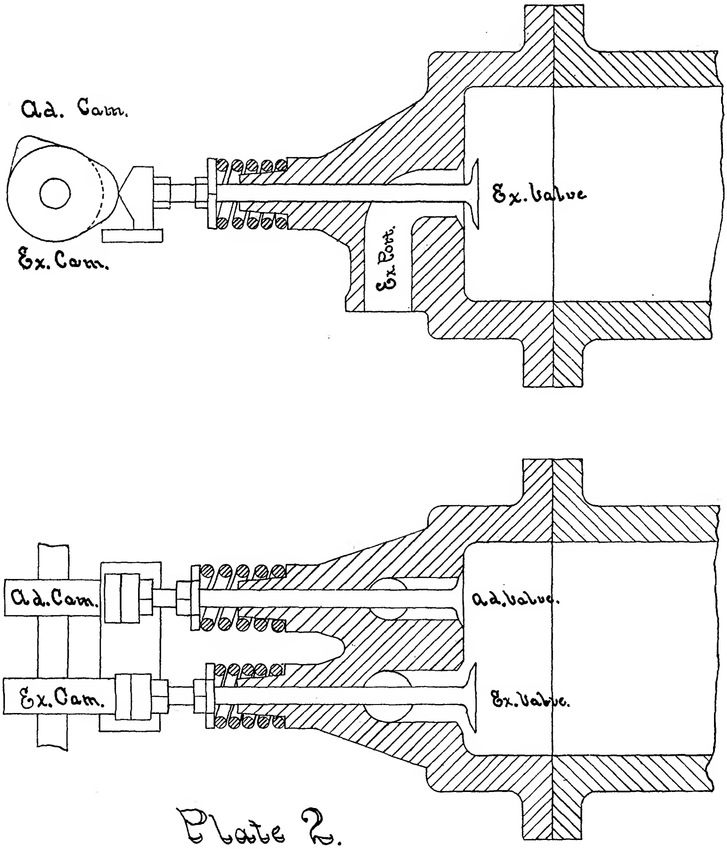


Plate 2.

Valve Gear
of
Cycle Gas Engine

piston stops is clearly shown. This card was taken while setting the valves, and is valuable only as showing the end of the admission stroke. Comparing cards 4 and 6, it will be found that *a g* in No. 6 and *a c* in No. 4 are equal, thus showing perfect action.

After making the last change, the engine was started, and card No. 5 was taken. This is an almost perfect theoretical card, and should show a high efficiency. A test made with the engine in this condition, shows a brake horsepower of 5 while the maximum indicated horsepower was 7.27 showing a mechanical efficiency of 69 per cent. The engine ran steadily at 80 revolutions per minute, with a pressure of 110 lbs. on the scale, the brake arm being three feet.

This is the minimum efficiency. As it was impossible to count the number of explosions, and as the intensity of the explosions varied considerably, the computation was made on the assumption of an explosion every revolution. The largest diagram on the card was taken, thus giving the largest possible indicated horse power, and the minimum efficiency. The real efficiency is much greater than this.

Our experience is an illustration of one of the many uses to which the indicator may be put, and is unique in that so far as we know, the indicator has never before been applied in this manner.

The results of our experiments seem to show that with positive ignition the engine should show a horse power greater than its rating, which is 10, while running at normal speed, which is about 130 revolutions per minute.

THE ARCHITECTURAL SCHOOLS OF THE UNITED STATES.

By Edward C. Earl, '94, Architectural Course.

The most important schools in the United States which have departments of architecture are the Massachusetts Institute of Technology, the University of Illinois, Cornell University, and the School of Mines of Columbia College. Other schools giving instruction in Architecture are: University of Pennsylvania (Philadelphia), University of Minnesota (Minneapolis), Case School of Applied Science (Cleveland), Art Institute of Chicago, Pratt Institute (Brooklyn), Brooklyn Institute (Brooklyn), Cooper Union (New York), and The Metropolitan Museum of Art (New York), besides several private schools.

Massachusetts Institute of Technology.

The Massachusetts Institute of Technology was granted a charter by the legislature in 1861; the architectural department was established five years later, though there were no regular graduates until 1873. Wm. R. Ware, an architect of Boston, and a former pupil of Mr. Hunt, was given charge of the department and continued at its head until he was called to the Columbia School of Mines in 1881. The present high standing of the school is largely due to his efforts. W. P. P. Longfellow and T. M. Clark, succeeded him as associate professors, but after one year Prof. Longfellow retired. Prof. Clark had worked under Richardson for several years and was employed in the building of Trinity Church, Boston, and several other of Richardson's large works. He has recently retired from the Institute and is now a practicing architect in Boston. Professor Francis W. Chandler is now at the head of the department. In 1872 Prof. Letang came from Paris and from that time until his death, (1892), had charge of the architectural drawing and designing.

Equipment. The department library contains 650 volumes, some 3 000 photographs, a collection of casts and models of architectural fragments, and a number of fine French school drawings. The library is freely accessible to students. Besides this special library, there are several thousand photographs, prints, drawings, and casts in the architectural museum; models and illustrations of architectural details and materials, arranged in the rooms of the department; a large part of the casts of architectural sculpture and details, in the museum of Fine Arts and a museum of sanitary and building appliances, mostly the donations of manufacturers.

Methods of Instruction. The Institute aims to prepare its pupils not only for their years of work as subordinates when accuracy, rapidity, and taste, in drawing and design, with knowledge of detail, will be the most useful qualifications; but also for their subsequent independent career, when the value of technical knowledge will become most important. The instruction comprises the study of building processes, and of professional practice, as well as that of composition and design, and of the History of Architecture.

The study of the Five Orders, and of Architectural History in the second year, is the beginning of strictly professional work. The instruction in Design is much the same as in the School of Fine Arts, Paris. An average of about fourteen hours per week for two and one half years is given to this work. The designs are criticised before the classes by a jury from the Boston Society of Architects. This Society has established two prizes of the value of $50, in books, for students who, at the end of the year, exhibit the best work.

The student is made familiar with the materials and principles of construction and the methods of heating and ventilating, by lectures and visits to buildings. Prof. Clark's *Buildiug Superintendence* is used in connection with the study of Specifications, Superintendence, etc.

The regular course of instruction occupies four years. Graduates receive the Degree of Bachelor of Science. The school year is divided into two terms of fifteen weeks each. A special course of two years is offered to draftsmen who do not wish to take the full four years course.

To be admitted to this course the student must be a college graduate, or twenty-one years of age with not less than two years office experience, and he must also pass examinations in plane geometry, and in free hand and mechanical drawing.

Admission. The candidate for admission to the Freshman class must have attained the age of seventeen years and must pass satisfactory examinations in Arithmetic, Algebra, Plane Geometry, French or German, English, and United States or Ancient History.

Fees. The tuition is $200 per year.

UNIVERSITY OF ILLINOIS.

The University of Illinois was opened in 1868 and has had a nominal department of architecture from the beginning. Mr. James Bellangee, a graduate of the Scientific department at Ann Arbor, had the direction of the department during the first two years. He was followed in 1871 by Mr. Harald M. Hansen, a graduate of the School of Architecture in Christina, Sweden, and a student, for two years, at the Bau Akademie in Berlin. In Sept. 1873 Professor N. Clifford Ricker came to the chair of Architecture. He was a graduate of the University and had studied abroad, having been a special student at the Bau Akademie. Prof. Ricker has ever since been at the head of the department and it is to his labors that the University owes its present place among the Architectural Schools of the country. Mr. James M. White was appointed Assistant in Architecture in 1891 and now has charge of Architectural Drawing and Construction.

Equipment. The University library contains about 750 volumes upon Architecture. The collection of photographs, prints, and engravings, placed in the architectural drawing room, numbers 12 000 cards and is at all times at the disposal of the students. In addition there is a collection of large architectural photographs and engravings, in the Art Gallery. The Mechanical Museum contains a collection of building materials, casts of ornament, and a number of drawings. The School of Art and Design has a good collection of photographs and casts of architecturel ornament, the collection of Moorish work being especially complete.

Methods of Instruction. Shop-practice is the only architectural work done in the first year. Two hours a day are devoted to this. "Shopwork was introduced into the University by Professor Ricker, in 1873, after having made an investigation of the Russian system as illustrated at the Vienna Exhibition." As the University of Illinois was the first institution in this country to teach shop practice the honor of introducing the Russian system of Manual Training into the schools of the United States is due to Prof. Ricker.

In the second year the study of building materials and construction is taken up by means of blue print lectures, problems and drawings being worked out by the class. In the Spring term Sanitary Science is studied.

In the Junior year the Five Orders are taken up, drawings made illustrating office work, and two terms devoted to Architectural History.

Architectural Design is first introduced in the Senior year. In that year Perspective; Estimates, Specifications, and Superintendence; Heating and Ventilation, and Designing of Ornament, are studied. In the second year three terms of optional work in free hand drawing, modeling, sketching, coloring, etc., are offered, and in the fourth year three terms are required.

Nine terms are devoted to Mathematics, pure and applied, but further changes are contemplated in the Architectural course so as to still farther differentiate it from the course in Architectural Engineering.

The school year is divided into three terms, one of fourteen and two of eleven weeks. Graduates are given the degree of Bachelor of Science.

Admission. Applicants for admission must be at least fifteen years of age and must pass examinations in the common branches and in Algebra, Plane and Solid Geometry, Physics, Botany, Physiology and Rhetoric.

Fees. Tuition is free. An incidental fee of $7.50 per term is charged to all University students.

CORNELL UNIVERSITY.

Cornell University was incorporated in 1865 and was opened in 1868. The Architectural Department was created in 1871 through the efforts of President White. Professor Charles Babcock has been at the head of the department from the beginning. He received his earlier Architectural training in the office of Richard Upjohn and was a partner with him for five years. In 1880 C. Francis Osborne was appointed Assistant Professor of Architecture. The work in applied construction and design is under the supervision of Prof. Osborne while Prof. Babcock has the general direction of the whole and has to do especially with the theory and history of Archtecture, Aesthetics, etc.

Equipment. The Architectural Museum contains about 1,500 photographs, several hundred drawings, and over 500 models in stone, wood and plaster, illustrating various constructive and ornamental forms of the various styles. The White Architectural Library contains over 1,000 volumes Both the Museum and Library are open at all times to students.

Methods of Instruction. History of Architecture and the development of the styles runs through five terms. The instruction is by lectures illustrated by photographs, engravings, drawings, slides, casts and models.

Office work is regarded as a necessary supplement to the course and the second year is so planned that the student may enter an office during the summer vacation after that year, special attention being given to construction, working drawings, specifications and contracts.

Design is first introduced in the first term of the Junior year. During this term special attention is given to house planning with illustrations drawn from the work of the best architects of the country. Problems in design, given out about once a week, are worked up at eight inch scale and criticised before the class. Exercises are given in sketching plans rapidly, forty minutes being allowed for the exercise, to develop the students capacity for getting his ideas into shape in the least possible time. In the third term of the Junior year a complete problem in design is worked out at eight inch scale; plans, elevations, sections, and perspectives, of some large building of a public class being required.

In the Senior year the work in design is intended to illustrate the historic styles. Roman, Gothic, and Renaissance buildings are worked up during the three successive terms.

Construction is taught chiefly by lectures illustrated by models.

The course extends over four years. Graduates receive the degree of Bachelor of Science. The school year is divided into three terms of eleven weeks each.

Admission. To be admitted to the Freshman year the student must pass examinations in English, Geography, Physiology and Hygiene, Arithmetic, Plane and Solid Geometry, Algebra, American History and French, German or Latin.

Fees. Tuition is $125 per year.

Comparative statement of the number of hours devoted to each of the following studies at the Massachusetts Institute of Technogy, (M. I. T.,) at the University of Illinois (U. I.) in Architecture (A.) and in Architectural Engineering (A. E.) and at Cornell University (C. U.)

STUDIES.	NUMBER OF HOURS AT DIFFERENT SCHOOLS.			
	M. I. T.	U. I.—A.	U. I.—A. E.	C. U.
Elementary Mathematics—(Algebra, Trigonometry)	110	120	120	120
Advanced Mathematics—(Analytical Geometry, Calculus)	110	240	240	110
Applied Mathematics—(Mechanics, Strength of Materials)	51	144	144	143
Materials and Construction	75	240	240	132
Elements of Drafting and Descriptive Geometry	295	360	360	110
Sanitary Science	15	60	60	0
Heating and Ventilation	30	60	60	33
Estimates, Business Relations, &c	90	180	180	11
Stereotomy	30	(In Mat'ls and Const.)	(In Mat'ls and Const)	22
Graphical Statistics	50	120	120, Elec. 120	0
Surveying	0	120	120	0
Hydraulics	0	0	84	0
Masonry Construction	0	0	84	0
Bridges	0	0	180	0
History of Architecture	105	240	240	187
Perspective	30	120	120 Elective	33
Architectural Design	1020	240	240	440
Freehand Drawing	570	360	0	220
Decoration		120	0	55
Photography	0	0	0	11
Landscape Gardening	0	0	0	22
Shop Practice	0	360	360	0
Physics and Chemistry	211	336	336	209
Language and Literature	405	288	288	165
General Science	0	0	0	77
History, Political Economy, &c	150	0	0	0
Military Science	90	168	168	110
Hygiene	0	0	0	55
Thesis	Included in Architectural Design.	No time assigned.	No time assigned.	44
*Total number of hours	3437	3876	3744	2265

THE SCHOOL OF MINES OF COLUMBIA COLLEGE.

The School of Mines of Columbia College established an Architectural Department in 1881. Its organization and control were in-

*Not including thesis.

trusted to Professor William R. Ware who has since continued at its head.

The Department has at all time had the generous support of the Trustees, besides which, one of their number, Mr. F. Augusts Schermerhorn has borne almost the entire expense of equipping the Department. In recognition of these benefactions the Trustees in 1889 set aside $13 000 of their invested funds to endow a Traveling Scholarship to be held by graduates of the Department. In the same year Mr. Charles F. McKim gave to the Trustees $20 000 for a similar purpose. These Scholarships are available in alternate years, two students receiving a thousand dollars apiece one year and one student receiving $13 00 the next. The choice is made by means of a competition in design held every winter.

The officers of instruction are as follows:

Wm. R. Ware, B. S., Professor of Architecture.

Alfred D. F. Hamlin, A. M., Adjunct Professor of Architecture; Ornament; History; Design.

Frank Dempster Sherman, Ph. B., Adjunct Professor of Architecture; Graphics; Elements of Architecture; History; Architectural Engineering.

Grenville Temple Snelling, B. S., Tutor in Architecture; Architectural Engineering; Design.

Maximilian K. Kress, Curator of the Collections; Archæology; Ancient History.

Charles A. Harriman, Instructor in Architectural Drawing.

George O. Tolten, Jr., A. M., Assistant in Drawing and Design.

Equipment.—The library contains about 700 volumes. It adjoins the drawing rooms and is at the disposal of the students at all times. The collection of photographs amounting to 12,000 is freely accessible. The books and photographs may be taken from the buildings. A number of water color drawings and a large number of architectural drawings made in the *Ecole des Beaux Arts* are used for copies and examples. The department has also a limited number of models and a collection of about 150 casts mostly presented by Mr. McKim. A classified collection of several thousand prints, engravings and wood cuts, serve as examples in History and Design. The College Library besides containing a large number of architectural books comprises the Avery Architectural Library which now contains 7,000 volumes and is constantly increasing. The Libraries as well as the library and drawing rooms of the Deparment are accessible to the students until eleven o'clock P. M.

Methods of Instruction.—Architectural work is begun in the first year. In that year the student takes up Ancient Architectural History, the Elements of Architecture, Projections, Shades and Shadows, Designing from description, and Free Hand and Architectural Drawing.

In the second year Perspective Problems in Design, and Modelling are added. The third and fourth years are always more technical.

Vacation work of some kind is required in all the courses. In the Architectural Department the student must present seventy-five sketches made during the summer, half of which are to be of architectural subjects. There are few restrictions as to the amount or kinds of sketching, but results show that there is no inclination to slight the work. The student is encouraged to spend his vacations in an architects office and each day so spent is counted in lieu of one sketch.

In the study of Architectural History it was found that the more carefully the lectures were prepared the less the books and photographs were studied. To remedy this in the study of the Gothic and Modern Styles, which are taken up in alternate years, the lectures are now condensed so as to occupy only half the year. During the second half year the books and photographs are divided into parcels, one of which is given to each student each week, to report upon. In this way each student goes through every book and looks at every photograph.

In the first term the Architectural Designing is done in some one of the classical styles. In the second term it conforms to the style that is being studied historically.

Free-hand drawing is practiced mainly with the pencil and pen. The students are also taught the use of the brush with india ink and with color.

French and German are taught by reading works upon Archaeology in class.

The course extends over four years. Almost all the class instruction is given in the first three years, the last year being mainly occupied with work in the drawing rooms. This year may be devoted to History and Design or to Construction and Practice, at the option of the student.

The school year is divided into two sessions of fifteen weeks each. The degree of Bachelor of Philosophy is conferred upon the graduates of the Department of Architecture. There are about 85 regular and 15 special students in the Department this year.

Post-graduate and university courses in history and design and in construction and practice are open to graduates of Colleges and Scientific Schools. Any men are admitted to these courses as special students who are qualified to pursue them. In general draftsmen who have spent three years in an architects office are considered qualified to follow the University course in History and Design. The course in Construction and Practice requires a knowledge of the higher mathematics. These special students are allowed to attend such other exercises as they have time for. "The Trustees have just begun to receive special students. They will probably continue to do so but to confine this privilege to men of considerable professional experience. Beginners are received only in the regular course, and however far advanced in this are not allowed to change and become special students except after a year's absence."

Admission.—Candidates for admission must be eighteen years of age, and must pass satisfactory examinations in Arithmetic, Geometry, Algebra, Trigonometry, Physics, Chemistry, German, French, English, Physical Geography, and Free-hand Drawing.

Fees.—The tuition is $2 00 per year.

The number of hours work is somewhat difficult to state but may be put very nearly as follows:

60 hours.	Advanced Mathematics (Analytical Geom. Calculus).
180 "	Applied Mathematics, Materials and construction. (Mechanics, Strength of Materials, graphical statistics, Engineering).
40 "	Sanitary Service.
10 "	Heating and Ventilation.
60 "	Stereotomy and Descriptive Geometry.—Shades and Shadows, etc. Surveying, (Optional during summer vacation).
200 "	History of Architecture.
440 "	Free hand drawing.
40 "	Perspective.
2 000 "	Architectural Drawing and Design.
120 "	History of Ornament.
270 "	Physics and Chemistry.
180 "	Language and Literature.
50 "	Hygiene.

3 650—Total number of hours.

THE BACTERIOLOGICAL EXAMINATION OF SEWAGE.

By R. J. Dickinson, '94, Sanitary Engineering Course.

The great obstacle in the study of bacteria is their extreme smallness. Because of this, one can only see, even with the best microscopes, the mere form, with often no distinguishing characteristics. For the same reason it is very difficult to seal the culture to outside bacteria and still allow free access of air. Such being the case too great care can not be taken to prevent contamination.

The method here recommended is merely an outline of the work, every step being hedged in by precautions, and even then the culture sometimes becoming contaminated.

In the examination of sewage two things are wanted: first, the number of bacteria in a given volume; second, their species.

To determine the number: Melt a tube of gelatin (any gelatin will liquefy at 40°C). Put into this a definite volume of the sewage to be examined and mix thoroughing by shaking. Pour out on a plate and cover with a bell-jar to prevent contamination. Place this culture in a temperature slightly below the melting point of gelatin—about 30° to 33°C. The first indication of the growth of the bacteria will be the appearance of small, unusually white dots. Each dot is a colony, reproduced from a single bacterium. When new-colonies cease to appear (usually at the end of two or three days), place a counting apparatus, a thin glass plate with the surface divided into small equal areas, over the culture and count the colonies in each of a number of these areas. From this counting estimate the total number of colonies, which is the number of bacteria in the volume examined.

The best ratio of the volumes of sewage and gelatin can be determined by experiment. It varies between very wide limits, the object being to have as much sewage as possible without having the colonies so numerous as to make counting difficult.

To determine the species it is necessary first to obtain a pure culture. This may be done as follows: Transfer from the plate-culture to a tube of distilled water, as small a quantity as possible from the colony whose species is to be determined. Mix thoroughly by shaking. From this tube inoculate a tube of melted gelatin.

Place this tube on ice or in a stream of cold water and roll it rapidly, so that in cooling the gelatin will form a thin film on the sides of the tube. This gives the same isolation of colonies as the plate-culture with much less danger of contamination, and hence is preferable. On account of the comparatively few bacteria transferred from the culture to the water, this inoculation will contain a correspondingly small number of bacteria. Hence there will be but little chance for two colonies to grow together as might be the case in the plate culture; therefore any colony will almost certainly be from one bacterium and consequently of one species. As soon as a colony is visible in the tube, inoculate a tube of bouillon from it. In most cases twenty-four hours will show a growth in the bouillon. When the growth appears, transfer some of it to a slide and examine it with a high-power microscope. If no difference in appearance can be distinguished it is called a pure culture. If differences, not due to known different styles of growth, are distinguished, the culture is not pure and the process must be repeated. If the culture is pure, inoculate tubes of bouillon, agar, gelatin, and milk. The appearance of the growth, the effect on the culture media, and the size and shape of the organism are the data by which the species is determined.

The bouillon used is made from beef. In making the solid medium, agar, bouillon is solidified by the addition of agar-agar (a species of gelatinous algæ). In making gelatin, use the best French gelatin instead of agar-agar. Many other subtances, such as blood-serum, potato, fruits, etc., are sometimes used, but those first named are by far the most common.

The media are generally used in test tubes which are filled about one third full and closed with tight cotton plugs.

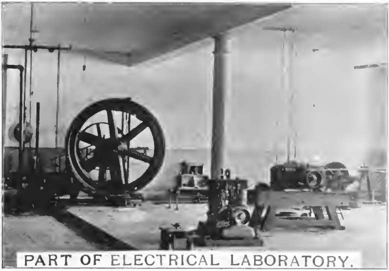
For methods of preparing these media, growing the bacteria, and details of sterilizing, the reader is refered to Bacteriological Technology by Salomousen, or Manual of Bacteriology by Crookshank. For a key by which to determine species see the work on Micro-organisms by Flugge and Report the Massachusetts State Board of Health for 1890.

NOTES ON THE OVERFLOW OF THE ILLINOIS CENTRAL R. R., IN LOUISIANA.

BY E. O. STREHLOW, '94, CIVIL ENGINEERING COURSE.

During the exceedingly high stage of the Mississippi river early in the spring of 1892, a break of some two thousand feet occurred in the Belmont levee about fifty miles above New Orleans, causing the overflow of hundreds of square miles of the Louisana swamps through which the I. C. R. R. passes. Immediately after the break began a large force of men was engaged in an effort to repair the levee, but without success. All available forces were then employed in the protection of the roadbed and in the increase of existing waterways by raising the bridges. Sacks filled with clay or earth were placed at the ends of bridges, along the slopes of embankments, and at the ends of ties, to afford protection from the scour. Brush was laid on the slopes and weighted down by old rails.

To preserve the line where the track was submerged two methods of anchoring were used; 1, stakes about three feet long, were driven on each side and near each end of the ties; and 2, a long iron rods shaped like the figure 7 so as to fit the top and sides of the tie, was driven at each end of a tie. The results obtained in the use of these methods were very satisfactory. About fifty miles of track was submerged, the maximum depth of water being thirty-one inches above the top of the rail at the lowest point in the track. Considerable damage to the rails resulted from the traffic which was maintained during the overflow. Freight trains were run continuously by substituting wood for coal; passenger trains were run over the submerged track for some time but were discontinued before the water reached its maximum elevation. The approximate cost of raising the track was $66,000 and the renewal, repairs, and raising of the bridges cost about $20,000 more.



PART OF ELECTRICAL LABORATORY.

MECHANICAL SHOP.

NOTES ON THE ROAD QUESTION.

By E. E. Barrett, '93, Civil Engineering Course.

A great deal has been said concerning the roads of Illinois, but in the voluminous literature on the question nothing has ever been given as to the amount actually collected for road improvement. By correspondence with a large number of county clerks in all parts of the state, an endeavor has been made to obtain the approximate amount collected in Illinois per year for road purposes.

The amounts collected in the counties given in Table I below are assumed to be fairly representative of that collected in the various counties throughout the state.

TABLE I.

Road and Bridge Tax Collected in 1892 in Ten Representative Counties of Illinois.

No.	Counties.	Area in Square Miles.	Cash Tax.	Tax Per Square Mile.	Average of Rates Per $100 in the several T'w'p's.	Labor Tax.
1	Champaign	976	$43 270.97	$44.33	54.2 cts.	$6 106.07
2	Christian	700	25 009.27	35.71	35.3 "	5 980.72
3	Douglas	410	25 025.52	61.03	60.5 "	....
4	Madison	696	36 558.91	52.52	42.3 "	....
5	McDonough	561	....	....	41.0 "	....
6	McLean	1 154	56 767.59	49.92	44.5 "	....
7	Pike	800	23 002.02	26.25	41.8 "	....
8	Rock Island	414	11 867.73	28.66	51.75 "	....
9	Stark	280	17 872.30	63.83	57.1 "	....
10	Whiteside	675	36 238.68	53.68	46.5 "	....
			MEAN	$46.20	47.5 cts.	

The writer was unable to obtain the amount of labor tax collected in 1892 in any of the counties excepting Champaign and Christian, as shown in Table I. These counties probably represent very closely the amount of labor tax collected in the various counties of the state. If the labor system of taxation could be entirely done away with and the road tax collected in money only, much more good could be accomplished, even with the limited means now available for road improvement.

In 1892 the State Board of Equalization gave $831 310 306 as the equalized value of assessable property in Illinois. Assuming 47.5 cents per $100, the average of the several counties in Table I, as the average rate of road and bridge tax, we have $3 948 724 (47.5 cents per $100 of $831 310 306) as the amount of cash collected for roads and bridges in 1892.

Of the cash tax a very large proportion is spent for bridges. In order to determine an approximation to this amount correspondence was had with one of the county supervisors of Champaign county who has been a member of the bridge committee of the County Board for a great many years and has had a wide experience. He says that one half of the road and bridge tax is spent for bridges and culverts.

In Illinois, county funds may be expended for bridges on county line; or, when for the past two years the town levy has been 60 cents per $100, the major part of which was needed for ordinary purposes, the county may contribute one half the cost of constructing a bridge; or, when the district has that year levied the full rate of $1, the major part of which is necessary for ordinary purposes, and when the cost of a bridge exceeds $100, the county may assume the total expense.

Table II below shows the actual amounts spent for bridges in all the townships in Champaign county securing aid in 1892. This does not include the amounts spent for bridges for which no county aid was asked.

TABLE II.

AMOUNT SPENT FOR BRIDGES IN 1892 IN TOWNS OF CHAMPAIGN COUNTY RECEIVING COUNTY AID.

No.	TOWNSHIPS.	ROAD AND BRIDGE TAX.	RATE PER $100.	COST OF BRIDGES.	AMOUNT PAID BY TOWNSHIP.	AMOUNT PAID BY COUNTY.
1	Compromise.	$2 446.70	60 cts.	$1 635.00	$817.50	$817.50
2	Crittenden..	1 072.22	40 "	849.00	424.50	424 50
3	Brown..................	1 858.54	60 "	1 740.00	870.00	870.00
4	East Bend.......	837.03	35 "	940.00	470.00	470.00
5	Homer.	3 034.75	100 "	860.00	430.00	430.00
6	Kerr..........	968.55	70 "	1 474.00	737.00	737.00
7	St. Joseph	1 927.78	60 "	1 100.00	550.00	550.00
8	Stanton....	1 507.54	60 "	1 625.00	812.50	812.00
		$13 655.11		$10 223.00	$5 111.50	$5 111.50

Since Champaign county has no very large streams and the topography of the county is such that no very damaging floods can occur in the streams to carry away or damage bridges, it may be taken as a fairly representative county. Therefore, assuming one half of the total amount of road and bridge tax collected in the state as spent for bridges, we have but $1 900 000 as the amount spent in road improvement proper.

The maximum amount of cash tax spent per square mile, or for two linear miles of road, was a trifle less than $64, or $32 per mile of road; while the *minimum* was only $13.12 per mile of road. The average amount was $23.10 per mile of road.

The amount of road and bridge tax per square mile, as shown in Table I, is strictly speaking more than the average amount spent on the two miles of road. In the smaller cities and towns a certain proportion of this tax is used on streets, thus making more than the two miles of road to each square mile of land. If half of the cash road and bridge tax is spent for bridges, which seems probable as shown above, we shall have a very small magin left with which to improve the roads. It is no wonder that our roads are in poor condition!

SHOP INSPECTION OF BRIDGES.

By C. O. Baughman, '94, Civil Engineering Course.

The term inspector is generally applied to a person employed by the engineer or the persons for whom the work is being constructed, having power to reject all work that does not comply in every respect with the specifications. (Such persons may be termed shop inspectors, but not in the sense that the term is used in this paper.) A shop inspector is a person employed by the manufacturers his position is in the shops and his duties are to see that all errors due to workmanship are corrected before the material is taken into the field. It is not only his duty to see that the work complies with the specifications, but that the painting is done properly, and that the material is loaded for shipment in such a manner as to be unloaded in the shortest possible time and at the least possible expense.

The method of inspecting the various parts of a structure will be briefly mentioned. First consider the floor beams and stringers, which are generally the first parts of a contract completed. See that all the rivets are tight. This is accomplished by tapping them lightly with a hammer, generally made for that purpose. If any loose rivets are found, do not have them cut out until their position in the piece has been carefully considered. Notice the plates that are riveted together. If they have been pressed closely together, have the rivet cut out and replaced by another. But if the plates do not appear very close together, most likely there is a burr formed around the rivet between the plates. Hence to back out a rivet of this kind would necessitate shearing off this burr, which in many cases proves to be more injurious than the loose rivet. Calking rivets is advisable only in such cases as this. Special care should be taken in the inspection of the riveting in large beams, especially plate girders where there are four or five cover plates riveted to the flange angles. If too many loose rivets are found, the foreman of the riveting crew should be notified, since they are often caused by the machine being out of adjustment, or the rivets not being of equal temperature when driven. Every riveted piece of a structure should be thus inspected.

Proceed with the inspection of the floor beam by examining the lateral hitches. See that the pin holes which take the lateral bolts are the right size and the proper distance apart. If the lateral system is composed of angles, be sure that the spacing of the field rivets is correct. Measure very carefully the spacing of the rivets in the web plate provided the stringers are connected to the web plate, and also the spacing in the end vertical angles of the stringers, and in the filler which goes between these angles and the web of the beam. If these measurements are not satisfactory, and you are not perfectly satisfied that the pieces will fit, you should have the pieces assembled and the holes reamed. Mark the pieces so that the erector may have no difficulty in putting them in the same position when erected. The stringers should be faced so as to make a flush joint with the beam. Measure their lengths; also apply a steel square at the ends to see that they are perpendicular with the flanges. The length of stringers with faced ends should check within one thirty-second of an inch. If the beam connections are made by riveting to the intermediate posts, their length should check within one sixteenth of an inch.

A steel square should be used to test all planed surfaces and also to test the perpendicularity of pin holes with respect to the axis of any section. Pin holes should be calipered and the distances from center to center should check within one sixteenth of an inch without regard to the distance. The four columns in the cantilever span in the bridge across the Mississippi River at Muscatine, Iowa, checked between centers of pins within one thirty-second of an inch in a distance of fifty-seven feet. If the pins are finished at the time you are inspecting the chord members, it is well to fit them into the holes and put on the nuts. Thus you are enabled to inspect both pins and pin-holes at the same time, and also check lengths of pins. Intermediate posts should check within one thirty-second of an inch between pin centers. Distance from center of pins to connection holes for knee braces should be correct; while the distance from pin center to sub-strut or longitudinal strut may vary even as much as one fourth of an inch and make no difference. Notice that the posts are in pairs with reference to these connections.

It was once customary to assemble all end top chord sections and end posts when the hip pin-hole was bored, and mark the several pieces so that they could be fitted together when the structure was erected. This took considerable time and has been almost done away with except where the specifications require it. When these pieces are bored separately the bevel should be checked, and the distance from the planed surface to the center of the pin hole should be measured on a line perpendicular to the planed surface, in both top chord section and end post, to ascertain the opening of the joint. In many cases the end sections of top chord as well as the end posts should be in pairs, the former with reference to top strut and top lateral connections and the latter with respect to portal and bracket connections.

Bottom chord bars and diagonals should check very closely, especially with respect to each other. Probably the best method for inspecting bars of this character is to place two pins of the required size the proper distance apart and have the bars placed on them. By this method the slightest variation in the length of a bar may be detected. All loop bars and rods, such as counters, top and bottom laterals, portal and sway rods, should be carefully examined about the loops to see that they have been properly welded. The lengths are not very important, as they are all adjustable and a half or three fourths of an inch either way makes no material difference.

In the inspection of roofs and general structural material the field connections are very important. In roofs where all the trusses are alike, one set of templates is used, and hence if there should be an error in any piece of a truss, it may possibly occur in the same piece throughout the entire roof. This same precaution should be used in the inspection of columns, since many times a great number are made from the same set of templates. It is customary to assemble one truss of a roof before it leaves the shops, thus avoiding all errors in lengths of pieces.

The painting of the various pieces is not very often considered, since many deem it of such little importance. Yet there are many parts of a bridge, more especially in columns, that are inaccessible to the painter after they have been assembled and riveted. The shoes of a bridge and bases of columns are examples. Hence these parts should be copiously painted when the pieces are being built.

Almost every car load of finished material should be examined. See that the pieces are in such a position with respect to each other that none will be strained by the motion of the train. If there are long sections, and it becomes necessary to load over the ends of the car, the pieces should be placed far enough from the side of the car that there will be no danger, when rounding a curve, of the ends projecting too far over the side of the train. If the pieces are very long and a bearing on two cars is necessary, swinging bolsters are required in almost all cases. Be sure that the timber of which the bolsters are made is of sufficient cross section, so that the ends will not have a bearing on the car when the load is upon them. They should turn freely, since a refusal to turn often results in derailment of the car or the falling off of the load.

In considering what has been said concerning the duties of a shop inspector, it is not to be understood that he must be able to execute all branches of the work himself, but he should have a sufficient knowledge of the various departments to be able to judge whether the work is or is not as it should be. There are many inspectors who have only a knowledge of a drawing, they are able to tell whether a thing looks like the structure or not; but as regards a knowledge of the workmanship and ability to judge of the quality of work that a shop with certain facilities ought to turn out, they are often deficient. Hence there are many embarrassing positions, not only for the inspectors but for the manufacturers also. It is to be hoped that in the near future the duties of a shop inspector may be considered of more worth than at present, so that experienced and competent men may aspire to the position.

A PECULIAR INSTANCE OF GRADE REDUCTION.

BY F. K. VIAL, '85, ASST. CIVIL ENGINEER BELT RAILWAY, CHICAGO.

My attention was recently called to what at first appeared to be, if true, a peculiar instance of grade reduction. It occurred on the Belt Railway of Chicago.

The road was built to furnish a simple and rapid means of transferring freight cars from one road to another. The general surface of the ground upon which the road-bed rests is practically level, but it was necessary to carry the grade over the important railroads, and as is usually the case with new roads, the grades were rather heavy. The traffic has become heavy and trains of great length are hauled. The ruling grades are of course the approaches to the viaducts, one of which is over the C. B. & Q. R. R., at Hawthorne. The south approach to this viaduct was originally a 1% grade, but was recently changed to a 0.54% grade, when it was found that there was no apparent benefit to operation, but rather a detriment, inasmuch as the change necessitated a reduction in the length of train previously handled.

It is my purpose to analyze the problem in a general way and to show that the original grade line was an easier operating grade, for long trains, than the new grade. It is not my intention to arrive at exact results, but only to show the reason why the new grade does not benefit operation in proportion to the change. For convenience of calculation I will use only round numbers and will not adhere to the exact profile.

It will answer our purpose to suppose a grade, B C, Fig. 1, page 30, 2 400 feet long rising 24 feet above the horizontal, with 100 feet level at the summit, CD, and then a descent by a 1% grade to the original level.

The grade on the south side was changed to about 0.5%, so we will call the new grade 4 800 feet long, at the foot of which the Chicago, Madison and Northern Railway crosses at grade, as shown in the diagram.

Suppose a train of 100 empties, weighing 18 tons per car, has come to a stop for this crossing. Let us find how much power would be required to carry it over the *old* grade. It has been shown by experiment that freight trains require about 6 pounds tractive force per ton to overcome the resistance of friction on a level track, and as the train begins to ascend a grade the resistance increases according to the well known laws of the inclined plane. Thus, the increased resistance per ton due to a 1% grade is 2 000 pounds × (1 ÷ 100) or 20 pounds (considering the track to be the same length as the the horizontal), and on a 0.5% grade the increased resistance per ton is 2 000 × (0.5 ÷ 100) or 10 pounds.

Suppose the tractive power of the locomotive to be 22 000 pounds. According to our supposition there were 2 400 feet of level grade on the old track between the point where the train stands at A and the 1% grade at B. The total weight of the train would be 100 × 18 tons = 1 800 tons, and 6 pounds resistance per ton would require 10 800 pounds of the power of the locomotive to overcome the resistance of friction, leaving 11 200 pounds to give motion to the train, making itself manifest by the constantly increasing velocity. All the power thus stored in the train is returned to assist the engine at critical points.

It will first be necessary to find the point upon the descending grade, which must be reached by the locomotive before its tractive force will again equal the train resistance. Suppose the cars to be 36 feet long, then the train length will be 3 600 feet. When the last car of the train reaches the foot of the 1% grade at B the engine will be 1 100 feet from the summit, D, on the descending side. Since the weight of the train is one half ton per foot, there would be 1 200 tons on the grade, 50 tons on the level CD, and 550 tons on the descending grade. The resistance for this position would be

$$10\,800+(1\,200\times20)-(550\times20)=23\,800 \text{ pounds,}$$

or 1 800 pounds more than the engine can pull; therefore enough more cars must pass the summit to overcome this deficiency, and

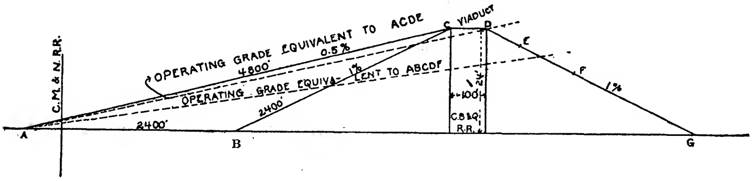

C.M.&N.R.R.
OPERATING GRADE EQUIVALENT TO ACDE
4800'
0.5%
OPERATING GRADE EQUIVALENT TO ABCDF
1%
2400'
2400'
VIADUCT
C
D
E
F
1%
24'
100'
C.B.&Q. R.R.
A
B
G

Fig. 1.

for every car that passes the summit there is one less on the grade BC; and as each ton on the descending grade, DE, returns 20 pounds, there would be 40 pounds less resistance for each ton that passes to the descending grade. It would be necessary for (1 800 ÷ 40) = 45 tons more to pass the summit before the power of the engine would just equal the train resistance. There would now be 595 tons on the descending grade and 1 155 tons on the ascending grade. Let us see how many foot pounds of energy it would require to move the train from its standing position at the crossing A to the above position.

For the first 2 400 feet, AB, the resistance would be uniform and to overcome it would require

$$2\,400\times 10\,800=25\,920\,000 \text{ foot pounds.}$$

For the next 2 400 feet, BC, (until the engine reaches the top of grade) the resistance would increase uniformly from 10 800 to

$$10\,800+(1\,200\times 20)=34\,800 \text{ pounds.}$$

The average per foot for this distance would be $(10\,800+34\,800)\div 2$, which multiplied by $2\,400=54\,720\,000$ foot pounds, which is the energy required. For the next 100 feet, CD, there would be a uniform resistance of 34 800 pounds requiring 3 480 000 foot pounds of energy. For the next 1 100 feet, DF, the resistance would decrease uniformly 10 pounds per foot, the average resistance per foot being $\frac{(34\,800+23\,800)}{2}$ which multiplied by $1\,100=32\,230\,000$ foot pounds.

It is necessary that 45 tons more pass to the descending grade before the engine can handle the train. The resistance per foot for the distance of 90 feet is $\frac{23\,800+22\,000}{2}$ which multiplied by $90=$ 2 061 000 foot pounds.

The total work required to move the train from the starting point, A, to the position where the resistance just equals the power of engine is

$$25\,920\,000+54\,720\,000+3\,480\,000+32\,230\,000+2\,061\,000$$
$$=118\,411\,000 \text{ foot pounds.}$$

The total distance traveled by the engine is

$$2\,400+2\,400+100+1\,100+90=6\,090 \text{ feet.}$$

The engine has then exerted energy equal to $6\,090\times 22\,000=133\,980\,000$ foot pounds, or 15 566 000 foot pounds more than the work of train resistance over this distance.

Now had the grade been level the train resistance would have been

$$6\,090\times 10\,800=65\,772\,000 \text{ foot pounds,}$$

and this subtracted from 118 411 000 leaves 52 639 000 foot pounds of resistance due to grade; this divided by the total weight of train in pounds will give the actual amount the center of gravity has been raised.

$$52\,639\,000 \div 3\,600\,000 = 14.62 \text{ ft.}$$

$$14.62 \div 6\,090 = 0.24\ \%\ \text{grade.}$$

This is if a grade line extended from the center of gravity of the train when the engine is at A to the center of gravity when the engine is at F, the rate 0.249 would be equivalent to the grade line, ABCD, to the point F.

But this assumes that A is raised and that the whole train is on the grade. Considering the train as it actually is on a level, since for the first train length the effect of the operating grade varies from 0 at starting to the full amount when the engine is 3 600 feet from A, the equivalent operating grade is $\frac{1}{2} \times 3\ 600$ shorter than 6 090 feet. Hence the operating grade is $\frac{14\ 61}{4290} = .34\%$. If the train were to be carried beyond F, the grade would evidently be less.

Now let us analyze the new grade line. As before, we will fix the point at the summit where the train resistance just equals the tractive force of the locomotive. We have the train resistance for a level grade 10 800 pounds and the resistance on this line due to grade is 10 lbs. per ton, or 18 000 pounds, which, added to 10 800 pounds, gives the total train resistance 28 800 pounds. It is therefore evident that a portion of the train must be on the descending grade before the engine could handle it. When the engine reaches the point D, there would be 50 tons less upon the grade, and the resistance reduced to 28 300 lbs. For every ton that passes to the descending grade the train resistance is reduced 30 pounds—hence 210 tons must pass the summit before the train resistance would be reduced to 22 000 lbs., making 420 feet from D to E. The total resistance from the point A to this point E would be

$$\frac{10\,800+28\,800}{2} \times 3\,600 + (1\,200 \times 28\,800) + \frac{28\,800+28\,300}{2} \times 100 +$$

$$\frac{28\,300+22\,000}{2} \times 420 = 119\,258\,000 \text{ foot pounds.}$$

and the distance traveled is 4 800+100+420=5 320, an average resistance of 22 417 pounds of which 11 617 pounds are due to grade. This corresponds to a grade of 0.323 %, which multiplied by 5 320=17.17 feet, the amount the center of gravity of the train had been raised in a distance of 5 320 feet.

Computing a grade line from A to the point E, since as before the equivalent grade line will be for a half train length less than the full distance 5 320 feet, we have $\frac{17.17}{3520}=0.487$ %.

It is therefore evident that the operating grade for the original grade line was 0.1 % easier than the new.

These results will show that the actual profile bears but little resemblance to the virtual or operating grade line. Hence the importance of fixing the operating profile before attempting to change an existing grade line, especially a momentum grade, i. e., a grade where momentum is depended upon to overcome the resistance in excess of the tractive force of the locomotive.

There are conditions which change the result; e. g. if the train was not required to stop at the crossing (which has an interlocking appliance), the two virtual profiles would be more nearly alike. Also the shorter the train, the nearer the profiles would correspond, until the limit is reached when the whole load is concentrated at one point. In this case all possible profiles between A and G would be identical so far as operation is concerned.

The assumption that the train load is equally distributed is not strictly correct but approximates closely enough for these notes.

It is also evident that not only has the grade line A, B, C, D not been improved but the grade line in the opposite direction G, D, C, B, A has also been increased for the center of gravity must be raised higher than on the original grade before the tractive power of the locomotive equals the train resistance. Going in this direction however the train has a longer stretch of level grade upon which to acquire velocity and seldom has difficulity in passing the summit, although the grade is about 1.2 %.

RELATIVE COST OF HEAVY VS. RE-INFORCED BRIDGES.

BY ALBERT F. ROBINSON, '80, ENGINEER BRIDGES AND BUILDING, C. R. I. & P. R'Y., TOPEKA, KANSAS.

Frequently an engineer must decide what is the heaviest bridge he can build with economy. In 1882 and 1883 the writer had charge of the designing and construction of some heavy iron bridges for one of our best western railroads. At that time the bridge companies bidding on these structures considered them unusually heavy; but during the past season several of these bridges were taken down and re-inforced because they were already considered too light for the service required. The writer has frequently found it necessary to urge the use of heavier bridges, and the data prepared for such discussions has suggested the following tables.

Tables I (page 73) shows a comparison between the cost of building a light bridge and afterwards re-inforcing it, and the cost of building the heavier bridge orginally. Only single track structues are considered, though the same relative proportion of cost and weight doubtless exist in double track bridges.

The bridges built in 1882 were according to the Erie Railroad specifications of 1879, and the "heavy bridges" are in accordance with the Lehigh Valley Railroad specifications of late date. The actual "shipping weights" were used in nearly every case. The prices, 4.7 cents and 5.1 cents per pound for the work erected and painted, are the lowest of which the writer has any record for the year 1882. During the year just passed the prices for bridge work have been about 3.75 cents per pound for griders up to 80 feet and 4 cents per pound for truss spans of ordinary length. These prices include erection, painting, and freight to Chicago, Ill.

The reasons for taking 15 years as the period for comparison are: 1st, It is almost 15 years since the first "Erie" specifications were published (about the limit of our observation for purposes of comparison). 2d, A large portion of the bridges which were fully up to the requirements at the time they were built, have had to be re-inforced within less than 15 years. 3d, It is hardly safe to predict the probable direction or rate of increase in the rolling load for a greater period than 15 years.

In Table I no charge has been made for freight between the bridge and the shops, for the delay or interruption of traffic on account of the work, or for the danger from floods when the track is

TABLE I.

Relative Cost of Constructing an Ordinary Bridge in 1882 and Re-inforcing it After 15 Years, and of Constructing the Heavier Bridge Originally.

ITEMS OF COST.	Span, in Feet, Between Pin Centers.								
	40	50	60	80	100	120	140	160	180
Cost of bridge in 1882 @ 4.7 cts. per lb.	$ 901.10	$ 1344.20	$ 1816.20	$ 3036.20					
Cost of bridge in 1882 @ 5.1 cts. per lb.					$ 4539.00	$ 6018.00	$ 7792.28	$ 9929.70	$11679.00
Additional increase in weight to have made the bridge safe for the heavier loading	9.5%	16%	14%	17.5%	15.75%	23%	22.7%	21.6%	22.25%
Additional cost to have made bridge safe for the heavier loading	$ 86.20	$ 216.20	$ 253.80	$ 531.10	$ 714.00	$ 1377.00	$ 1744.20	$ 2148.10	$ 2601.00
Amount of this additional cost, in 15 years, at 5% compound interest	179.30	449.70	527 90	1104.70	1485.12	2864.16	3627.94	4465.97	5410.08
Cost of re-inforcing after erection:									
Material and shop work @ 3.25 cts. per lb	$ 59.60	$ 149.50	$ 175.50	$ 367.25	$ 455.00	$ 877.50	$ 1111.50	$ 1368.25	$ 1657.50
Taking down and loading bridges @ 0.25 and 0.30 cts. per lb., respectively				161.50	222.50	354.00	458.40	584.10	687.00
Erecting and painting @ 0.5 cts. per lb.	105.67	166.00	220.00						
Erecting and painting @ 0.6 cts. per lb.				455.40	618.00	870.00	1122.00	1428.00	1680.00
Lower false work for carrying trains	212.00	265.00	318.00	424.00	630.00	756.00	882.00	1008.00	1134.00
Total cost of re-inforcing	377.27	580.50	713.50	1408.15	1925.50	2857.50	3573.90	4388.35	5158.50
Proportion of cost of re-inforcing to first cost of bridge.	41.4%	43%	39.25%	46%	42%	47%	46%	44%	44%
Amount which at 5% compound interest will in 15 years equal the cost of re-inforcing	$ 181.38	$ 279.09	$ 343.03	$ 677.00	$ 925.72	$ 1373.80	$ 1718.22	$ 2109.80	$ 2480.05
Proportion of this amount to first cost of bridge	20%	21%	19%	22%	20%	23%	22%	21%	21%
Additional cost in 1882 of making bridge safe for the heavier loading	$ 86.20	$ 216.20	$ 253.80	$ 531.10	$ 714.00	$ 1377.00	$ 1744.20	$ 2148.10	$ 2601.00
Amount which at 5% compound interest will in 15 years equal cost of re-inforcing	181.38	279.09	343.03	677.00	925.72	1373.80	1718.22	2109.80	2480.05
Amount which at 5% compound interest will equal at the end of 15 years the extra cost of making the bridge safe for the heavier loading	$ 179.30	$ 449.70	$ 527.90	$ 1104.70	$ 1485.12	$ 2864.16	$ 3627.94	$ 4465.97	$ 5410.08
Total cost of re-inforcing the bridge	377.27	580.50	713.50	1408.15	1925.50	2857.50	3573.90	4388.35	5158.50

carried on temporary false work. Taking these items into the account, we may conclude that at any time during the past 15 years, the heavier bridge would have been the cheaper.

Under "Cost of re-inforcing bridges after erection," (see the second portion of Table I), the price per pound for the new part of the work has been taken at 3.25 cents, not including cost of erecting and painting. New bridge work is 3.25 cents per pound, excluding cost of erecting and painting, but re-inforcing old work is always more expensive per pound of the new material added than entirely new bridge work.

The cost of lower false work for carrying teams and supporting iron and upper false work during erection, was taken from Table II.

TABLE II.

COST OF FALSE WORK FOR CARRYING TRAFFIC.

ITEMS OF COST.	SPANS LESS THAN 100 FEET.		SPANS 100 TO 200 FEET.	
Cost of new false work per lineal foot of track		$8.00		$9.50
Relaying track after putting in new floor*		.30		.30
		$8.30		$8.30
Value of false work after use, per lineal foot of track	$4.00		$4.50	
Cost of removing false work	1.00		1.00	
Net value of old false work, piled on bank ready for loading	$3.00	$3.00	$3.50	$3.50
Net cost of false work per lineal foot of track		$5.30		$6.30

*This does not include the cost of new bridge ties or their framing.

In computing Table II, the caps were assumed to be double or treble, and the lower or main caps not less than 22 feet long. Sway bracing and piling are used as in ordinary pile bridges, but the piles are frequently of pine or some cheap timber. The stringers are not usually bolted together, but are so arranged that they can be easily removed.

On a road built in a large river valley where side streams had to be crossed near their mouths, the total pile and timber trestle bridging averaged about 280 lineal feet per mile of road. The average cost of this bridging was a little more than $10 per lineal foot. On a line in Kansas and Colorado, averaging less than 100 feet of bridging per mile, the cost was from $10.50 to $13 per lineal foot of bridge. In view of these facts the prices assumed for false work can hardly be considered as too high.

TABLE III.

RELATIVE COST OF CONSTRUCTING AN ORDINARY BRIDGE IN 1892, AND RE-INFORCING IT AFTER 15 YEARS, AND OF CONSTRUCTING THE HEAVIER BRIDGE ORIGINALLY.

ITEMS OF COST.	SPAN, IN FEET, BETWEEN PIN CENTERS.								
	40	50	60	80	100	120	140	160	180
Cost of ordinary bridge in 1892 @ 3.75 cts. per lb.	$ 792.54	$ 1244.00	$ 1650.00	$ 2846.75	$ 4120.00	$ 5800.00	$ 7480.00	$ 9472.00	$11200.00
Estimated increase in weight (see Table II)	9.5%	16%	14%	17.5%	15.75%	23%	22.4%	21.6%	22.25%
Additional cost of above increase in weight	$ 158.51	$ 248.80	$ 313.50	$ 569.35	824.00	$ 1276.00	$ 1645.60	$ 1989.12	$ 2352.00
Amount of this additional cost in 15 years @ 5% compound interest	329.70	517.50	652.08	1184.15	1713.92	2654.28	3422.85	4137.37	4892.16
Cost of re-inforcing after erection:									
Material and shop work @ 3 cts. per lb.	$ 126.81	$ 199.20	$ 250.80	$ 455.40	$ 618.00	$ 957.00	$ 1234.20	$ 1491.84	$ 1764.00
Tearing down and loading @ 0.25 and 0.3 cts. per lb. respectively				189.75	309.00	435.00	561.00	710.40	840.00
Erecting and painting @ 0.5 and 0.6 cts. per lb. respectively.	126.81	239.04	261.80	455.40	741.60	1061.40	1368.84	1719.17	2032.80
Lower False works for carrying trains.	212.00	265.00	318.00	424.00	630.00	756.00	882.00	1008.00	1134.00
Total cost of reinforcing	465.62	703.24	830.60	1524.55	2298.60	3209.40	4046.04	4929.41	5770.80
Additional cost in 1892 of making bridge safe for the heavier loading.	$ 158.51	$ 248.80	$ 313.50	$ 569.35	$ 824.00	$ 1276.00	$ 1645.60	$ 1989.12	$ 2352.00
Amonnt which at 5% compound interest will in 15 years equal cost of re-inforcing.	223.85	338.09	399.32	732.95	1105.10	1542.98	1945.21	2369.90	2774.42
Amount which at 5% compound interest will in 15 years equal the extra cost of making the bridge strong enough for the heaviest traffic	$ 329.70	$ 517.50	$ 652.08	$ 1524.55	$ 1713.92	$ 2654.28	$ 3422.85	$ 4137.37	4892.16
Total cost of re-inforcing the bridge.	465.62	703.24	830.60	1184.15	2298.60	3209.40	4046.04	4929.41	5770.80

Judging from the past it is safe to assume that the rolling load will contine to increase for some time to come. The reasons for this, are: 1. This increase may not be proportionally so great as in the past, but the uniform train loading may increase much faster than the concentrated engine wheel loadings. 2. Even though steam be superseded as the motive power, it is hardly probable that the maximum car loads will be reduced; on the contrary this may result in increasing them. It is not likely that the demand for cars to carry extra heavy pieces of machinery will cease. This demand has steadily kept pace with the increased strength of rolling stock and permanent way; and has in fact generally been the chief cause for these increases. 3. Competition and the demands of the traveling public have been continually for a higher average speed for trains of all classes. At present it is not uncommon for special classes of valuable freight to be carried at express train speed. The higher rate of speed for all classes of business must cause greater wear on bridges and this will naturally call for heavier structures.

With the prevailing tendency towards heavier service in view, the writer has prepared Table III, page 75, which gives the cost of heavy bridges at present market prices and shows the percentage that can be added to the weight (when the structures are built), without having the extra cost amount to more than the prospective cost of re-inforcing after 15 years.

ENGINEERING MUSEUM.

PART OF MINING LABORATORY.

HEATING AND VENTILATION OF RESIDENCES.*

By James R. Willett.

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Young gentlemen,—I have been invited to speak to you on the subject of heating and ventilation, in its practical applications. The theory of the subject, I presume you have been, or will be, instructed upon by others. My purpose is to show you the way in which a person engaged in such business may prepare designs for executton.

Many years ago, long before you young gentlemen were born a young architect had prepared designs for a residence, and had acquitted himself to his entire satisfaction; and, as he thought, to the satisfaction of the owner. One day, when the building was nearly completed, the owner asked the architect, "What size furnace would you use in the house?" For in that day, heating by hot water and steam, at least as applied to residences, was uncommon. The young architect could not answer the question, for he knew nothing about it. However, he said he would consider the matter, and in that way postponed the decision.

How was he to ascertain the proper size for the furnace? He could find books of the subject of heating and ventilation, but they did not enable him so arrive at a prompt and definite answer to the question asked.

The young architect at last found that the only way to acquire knowledge on the subject was to go to the contractors who were engaged in putting up furnaces. After acquiring all the information he could from them, comparing their statements, etc., he arrived at a conclusion which he gave to the owner in, perhaps, a somewhat oracular manner.

The statements made by the furnace men did not, in many instances, agree. Each man was, of course, interested in the sale of some particular furnace. They possessed a large amount of information derived from actual work, and they carried on the work largely by their "judgment." The only rule which appeared to be general was: They would take the volume of the building, and they knew, from previous experience, about such a size furnace would be sufficient for such a building. The experience they had acquired was very valuable, but it was not digested and arranged so that the general rules could be deduced and applied to other buildings.

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*Address delivered before the Engineering College of the University of Illinois, March 23, 1893.

We propose to give "office" rules—that is, such rules as can be promptly and readily applied in making out designs and specifications for such work. It is to be understood that these rules are approximate only. But they are sufficiently correct for practice, as experience has shown. Every practitioner has such rules, must have such rules, and every practitioner who has written on the subject has given such rules. The endeavor has been made to give rules so that the field for judgment will be limited, and to prevent grievous errors from being made; but judgment in the use of these rules is required. No rules can be made on any subject that do not require the use of the judgment in their application.

There are many methods or systems of heating and ventilation· There is no one that is, under any and all circumstances, superior to every other. This lecture will be confined to the heating and ventilation of ordinary residences in a simple manner. There are several reasons for this; one is that you must creep before you walk. If a young architect will master the lower walks of a subject, he will be afterward much better fitted to comprehend the higher branches of it. Besides the time allotted to this lecture will not admit of an extensive treatment.

Any residence which has a heating apparatus designed to be run in the best manner, and requiring thorough and constant attention, will be more or less of a failure. The boilers in an ordinary residence are usually run by the owner of the house, by a servant girl, or at best by a coachman whose time and attention is mostly occupied with other duties, and who looks upon the necessity of attending to the heating apparatus as an infliction and a burden; therefore, it is essential that in designing a heating apparatus for a residence, it shall be capable of working satisfactorily under some amount of neglect and misuse. This neglect is contemplated and allowed for in the statements hereinafter made.

The method of heating a building should be decided upon before the plans for the building are finished, as the method of heating affects, or should affect, both the plans and construction of the building.

VENTILATION.

To make a proper design for ventilation, it is necessary to know or to assume:

The number of persons who are to inhabit the house.

The fresh air that should be allowed to each person. This is put down by some authors as high as 2 000 cubic feet per person per hour, and even higher.

The quantity of air necessary to provide for the vitiation of the air by gas or other lights.

How can anyone tell what number of persons will occupy a residence and for how many hours per day? How often will the number of persons change? How many lights will be kept burning, and for how many hours will they burn? How much air will leak into the house or be forced in by the winds outside?

All of these questions are somewhat in the nature of conundrums. In the same residence the data will vary from day to day.

We must assume a certain data, depending upon the style of the house, the class of people who occupy it, etc.

It may be assumed that there will be two persons occupying each bedroom; and the number of bedrooms multiplied by 2 may be taken as the total number of persons to be provided for.

As to the lighting, that is even more uncertain than the number of persons in the house. It may be assumed as one burner per person. The number of hours that the lights are burning will probably not exceed six hours out of the twenty-four. Indeed, all the lights will not probably burn that long, and four hours out of the twenty-four may be sufficient, but we will consider them as burning six hours out of the twenty-four.

The air required for the vitiation by gas lights is given by Schumann as 1 800 cubic feet per hour for each cubic foot of gas burned. As each burner may be taken to burn four cubic feet of gas per hour, then each light would require 7 200 cubic feet per hour. This is, however, for but six hours out of the twenty-four. If then it be assumed that a burner will burn six hours, and distribute it over the twenty-four hours, it will give 1 800 cubic feet per hour. Adding 2 000 cubic feet, the quantity required for the use of each person, to 1 800 cubic feet, the quantity required per person for the consumption of one burner of gas, gives a total of 3 800 cubic feet of fresh air required for personal use, together with gas consumption, per person, per hour.

If we assume the velocity of the air entering the fresh-air duct at six feet per second, and it is usually as much as that, if the rules hereinafter given are complied with; then to give 3 800 cubic feet of air per person per hour will require a cross sectional area in the fresh-air duct of 25.4 square inches per person, or say 25 square inches per person.

The question of the leakage of the air into the building is a very considerable one. The more leakage there is, the less area will be required in the fresh-air duct (and vice versa to some extent). In the very best built houses the leakage is always large. There is no way of ascertaining its amount, but it is thought fair to assume that not less than one-fifth the volume of air that will be required in the house is furnished by leakage; and when a strong wind is prevailing it may be much more. It is true that there is leakage out of the house as well as into it; but the leakage out of the house is less than into it, because the outward leakage must pass through two walls instead of one and because the heating of the house draws air to it. It is thought that the difference will amount to one-fifth, as above stated.

If we take then one-fifth of the fresh air as furnished by leakage it will reduce the area of the fresh-air duct to twenty square inches per person. It is not advisable to make the area of fresh-air duct smaller than this, and it is much better to make it larger, provided always that the radiator, or warming, surface for heating the fresh air, is increased in the same ratio, for if the incoming air is cold it will be shut off.

The foregoing can be condensed to—

$$\left.\begin{array}{c}\text{Area of fresh-air inlet in}\\ \text{square inches,}\end{array}\right\} = 40 \left\{\begin{array}{c}\text{Number of bedrooms in}\\ \text{residence.}\end{array}\right.$$

On a windy and cold day, it may be found that it is impracticable to keep the house warm with the fresh-air duct fully open. Or, even if it could be kept open, the consumption of coal would be such that the owner would not incur the necessary expense.

Here it may be remarked, that the question as to how the owner is likely to act under certain circumstances is one that the practicing architect should take into consideration.

If the fresh-air inlet is large, and owing thereto, the house either becomes cold, or the amount of coal used is, in the judgment of the owner, excessive, it is practically certain that he will shut the fresh-air inlet up; probably shut it up altogether, and thereby deprive the house of all fresh air except what it may obtain through leakage. Long and continuous experience has shown this to be the case. Whereas, if there is a means of closing the fresh-air inlet to some extent, but not altogether, the owner is likely to so close it. This will, of course, decrease the supply of air, but it is better to decrease the supply of air than to altogether stop it.

It is well, therefore, to provide a damper in the fresh-air inlet. This damper, however, should not close the inlet wholly; say one-third of the damper should be cut off.

If the damper is thus arranged and the proportions given in the rules observed, the supply of fresh air will still be equal to 1 200 cubic feet per person per hour. Indeed, it will be more, since the leakage will not be diminished, but rather increased. Besides, when the wind is blowing hard the velocity in the fresh-air duct will be increased.

The volume of air in cubic feet that will pass through a duct of any cross sectional area, with air moving at any velocity, can be found by recollecting that a duct of 1 inch cross sectional area, with air moving at a velocity of 1 foot per second, will pass 25 cubic feet of air per hour. If, therefore, 25 be multiplied by the cross sectional area of any duct, in square inches, and also by the velocity in feet per second, the product will be the quantity of air in cubic feet per hour that said duct will pass.

Example: What volume of air moving at 8 feet per second will pass through a duct having a sectional area of 113 square inches, in one hour, $25 \times 8 \times 113 = 22\,600$ cubic feet.

The opening of the fresh-air inlet should be upon the side of the house against which the prevailing cold winds blow. (This means, in the vicinity of Chicago, first the north side and next the west side.) The openings should not be exposed to the direct force of the wind. If there are no buildings, fences, or other obstructions close by to prevent the wind from striking the opening directly, then some shield should be erected to do so. There should be but one opening to the fresh-air inlet; but if there are two boilers or furnaces to the house, then the openings to the fresh-air inlets to each should be on the same side of the house.

If there be fresh-air inlets on the different sides of the house, it is not at all unlikely that the fresh air will at times be driven in one inlet through the radiators or furnace, through the house and out of the other supposed fresh-air inlet.

The fresh-air duct should not be taken under the basement floor, where it is liable to fill up from flooding of sewers, dirt, dead rats and other filth; but it should be kept above the ground, brought along the basement ceiling until it reaches the proper point and then, and not till then (turning down if necessary), enter the bottom of the indirect radiator casing, or furnace.

All incoming fresh air must be heated by indirect radiator, or by the furnace.

The hot-air flues to rooms will be spoken of when speaking of indirect radiation hereafter.

The outlet flues through which air passes out of a room, and out of the building, should be of brick, similar to smoke flues, though they are sometimes made of tin. Every room should have such a flue. The opening from a room into outlet flue should be close to the floor, in, or just above, the baseboard. This opening should have a register. This register should be rather small than large, for if large outlet registers are used some of them are apt to become air inlets instead of air outlets—that is, some are apt to "draw downward," as it is termed, instead of upward.

A general rule for such an outlet register may be taken as follows:

For first-story rooms: Divide the volume of the room in cubic feet by 30, and the quotient will be the gross area of the outlet register in square inches.

For second-story rooms: Divide by 40.

For third-story rooms: Divide by 50.

The outlet register should not be much greater than called for by the above rule unless the fresh-air duct is enlarged.

The sum of the gross area of the outlet flues should not exceed $1\frac{3}{4}$ the area of fresh-air inlet duct. If it does, then either they should be reduced or the fresh-air inlet duct enlarged. The latter is best.

The fresh-air entrance to inlet duct should be covered with wire netting to keep out dust, etc., and this entrance should have at least twice the area of the inlet duct with which it connects, as the wire netting diminishes the supply of air. If this area cannot be had it is better to leave off the netting.

The rules given are based on fresh heated air being admitted to the house through an indirect radiator. If indirect radiation is not put in, then the direct radiation should be increased by one-half (or more) of the indirect radiation.

HEATING.

Residences are heated in three different ways:

By hot-air furnances;

By steam apparatus;

By hot-water apparatus.

How shall a residence be heated? This is a question which is largely determined by the owner; usually on the score of expense. The cost of the heating apparatus may be taken in the following order: The furnace is the cheapest, the steam above that, and the hot-water apparatus usually costing the most.

The buildings that are usually planned by an architect are not nowadays often heated by furnace. Nevertheless, most of the residences throughout the country that have any heating apparatus, are heated by furnace, and the young architect should know something about them.

In hot-water heating the air is often exposed to a temperature of but 100 degrees Fahr., and is not likely to be over 180 degrees. The temperature of the water is practically never above 212 degrees, and usually not more than 150 degrees.

In steam heating the air comes in contact with a surface which is not usually below 212 degrees Fahr., while it is often much higher, the air being exposed to a temperature which is seldom less than 200 degress and often as high as 300 degrees.

In hot-air furnace heating the method of heating is altogether what is termed the indirect system of heating. The incoming fresh air comes in contact with a surface highly heated. It is liable to be exposed to a surface which may have a temperature as high as 600 degrees, and which often has a temperature of 400 degrees.

The air which we breathe is full of particles of dust. Now, when this air comes in contact with a heated surface, such as it may do in a hot-air furnace, the dust is burned; and the air perhaps otherwise injured and rendered unfit for breathing. In heating by steam, the same thing may occur, but in a very much less degree. But in the open system of hot-water heating, it is practically impossible to run the temperature over 200 degrees, and it ordinarily does not exceed 160 degrees. In heating by steam, you cannot make steam at all without bringing the temperature to the boiling point of water (212 degrees) and therefore the temperature of the radiators must be nearly that height. Now, this may be too high for mild weather.

In the "open" method of hot-water heating you cannot raise the temperature above 200 degrees, and therefore you cannot burn the air; and you can regulate the heat to any lower temperature you desire, down to the temperature out of doors. These are the great advantages the hot-water heating system possesses.

In the selection of heating apparatus for residences preference should be given in the following order: First, hot-water; second, steam; third, hot-air furnace.

While the above is true, yet it is better to have a good apparatus of an inferior type than a poor apparatus of superior type. A residence can be heated and well warmed by any of them, when the apparatus is well designed.

Whatever system may be adopted, the following data are needed for each room to be heated: Volume in cubic feet; exterior wall surface in square feet; exterior windows and exterior doors in square feet.

In measuring exterior wall surface, include the windows and the doors areas. In measuring windows (and doors) measure between the inside jambs, that is include width of sash. This is to compensate for leakage of air. The exterior doors are treated as if they were windows in order to compensate for the air leakage during the opening and shutting thereof, etc.

Proceed by finding the value of R for each room according to the following formula:

I=Inside temperature desired Fahrenheit, usually 70 degrees.
E=Exterior temperature.
V=Volume of the room in cubic feet.
W=Exterior wall surface of the room in square feet.
G=Exterior windows and exterior doors in square feet.
R=Standard amount of surface of radiator in square feet.

$$\left(\frac{V}{400}+\frac{W}{10}+\frac{G}{2}\right)\left(\frac{I-E}{I}\right)0.9=R$$

In considering the exterior temperature is is not necessary to assume the very lowest that occurred. Such extreme temperatures usually last but a short time, and any full sized apparatus will bear pushing occassionally. If the lowest temperature is increased by ten degrees the result may be assumed as the exterior temperature. In Chicago the exterior temperature is usually assumed as—10 degrees, although it sometimes has gone down to—20 degrees and even lower.

The portion of the equation, $0.9\left(\frac{I-E}{I}\right)$ may be reduced to a constant for any particular place. "I" being taken as 70 degrees Fahr., and "E" as 10 degrees higher than the minimum temperature. It is convenient to have these constants, and they are given in Table I.

TABLE I.

Table of minimum temperatures, and the values of $\frac{I-E}{I}=\frac{70-E}{70}$ "E" being taken as 10 degress higher than minimum temperature.

PLACE.	Minimum Temperature. Degrees.	Value of $\left(\frac{70-E}{70}\right)0.9$
Duluth, Minn	—38	1.25
St. Paul, "	—32	1.18
Milwaukee, Wis.	—25	1.09
Burlington, Vt	—20	1.03
Chicago, Ill	—20	1.03
Detroit, Mich.	—20	1.03
Indianapolis, Ind	—18	1.00
Albany, N. Y.	—17	0.99
St. Louis, Mo.	—16	0.98
Buffalo, N. Y.	—13	0.94
Cleveland, O.	—13	0.94
Pittsburgh, Pa.	—12	0.92
Portland, Me	—12	0.92
Boston, Mass.	—11	0.91
Louisville, Ky	—10	0.90
Cincinnati, O	— 7	0.86
New York, N. Y.	— 6	0.85
Philadelphia, Penn	— 5	0.83
Washington, D. C	— 5	0.83
Baltimore, Md.	— 2	0.80

In places where the minimum temperature is above zero, residences are not usually heated by apparatus, but by grates.

For the proper amount of direct radiator surface in square feet necessary to be used in a room, use the following formula;

$Q=R\ F\ J.$

Where Q=Desired square feet direct radiator surface for a room.

R=Standard surface of radiator in square feet for a room calculated as heretofore given.

F=Factor depending on method of heating, or on particular story, etc.

J=Factor depending on exposure, etc.

If the house is to be heated by indirect radiation there must be hot-air flues provided to reach rooms in upper stories.

The tendency nowadays appears to be in the first story of residences to throw the rooms together by having large openings between themselves and the stair hall without doors. The wall space in the first story is thus reduced to a small amount. It is difficult to find room in it to contain the hot-air flues.

You have flues in the building, very large flues, sometimes more than one of them. It is true they are not called flues, but nevertheless they are flues, and, unless you need to throw a small amount of very hot air into a room, you can do away with the small hot-air flues and use the large flues already provided. These large flues are called stair halls. They have several advantages over the small flues. They can deliver a large volume of air at a velocity so slow as to be inappreciable. The air need not be heated up above a low temperature, say 80 to 90 degrees Fahr. In rooms where there are openings without doors, the air can pass. Where there are doors the air can enter by means of a low transom over the door.

One of the difficulties of heating by the indirect system of small flues is the fact that the large flue, known as the stair hall, is frequently working at cross purposes with the small hot-air flues. On a windy day it is certainly difficult to thoroughly heat, by the indirect system alone, that side of the house exposed to a gale. The exclusive use of the indirect system is not necessary for ordinary residences. The best result is obtained by a combination. With direct radiators for the warming of the room and indirect radiators for the warming of the neccessary incoming fresh air.

By direct radiation is meant that radiators are to be placed in the room which is to be heated. By indirect radiation, that radiators heat air and then this heated air is passed into the room.

Usually indirect radiators are placed in the basement just below the ceiling. This brings the bottom of the radiator down low compared with the boiler. Now the higher the radiators are above the boiler the better.

A radiator has been introduced in late years which is termed the direct-indirect. This radiator, to all appearnces, is a direct radiator, but the sides are partially closed in and a fresh-air duct is brought below it from which fresh air is received, and passed up through the radiator. This direct-indirect radiator can be placed on (and above) the first floor, and is well above the boiler which is usually placed in the basement, therefore it works better than if lower down. The opening in the floor through which the air passes up is covered by the radiator. We consider this radiator preferable to the old (or basement) indirect radiator.

The indirect system of radiation permits of ventilation, at least requires the circulation of air both in and out of a room, whereas the direct system causes the air already in the room to circulate in it, and does not require the introduction of air from outside of the room. Each system has its own benefits and its disadvantages. The indirect system makes it difficult at times to heat some of the rooms which are exposed to prevailing winds, and has been known to act in a reverse manner, driving the air in the upcast flue, or what is intended for such, downward. Furthermore, the quantity of heat necessary to warm the room depends upon the amount of air introduced at the proper temperature. Now, the quantity of air required for ventilation and that required to carry the necessary amount of heat, are not usually similar. The requirements of heating and of ventilating usually clash, and one or the other must be, to a more or less degree, sacrificed.

The indirect radiators should not have any valves upon them. If any heat at all is needed they should be in operation; besides, as they are exposed to the incoming fresh cold air, they are very liable to freeze up if the flow of steam or hot water is stopped.

In the direct system, the radiators being in the room to be heated, the question of the quantity of air introduced, so far as the warming of the room is concerned, nearly disappears, and a direct radiation can be proportioned to the necessary amount required to warm the room and to warm it only, while the ventilating of it can be treated separately and apart.

In large, expensive buildings there are sometimes built hot-air flues to each room, and if the building is to be heated solely by indirect radiation this is imperative; but if the rooms are already heated by direct radiation it is only necessary to heat the incoming fresh air that is required for ventilation, and this brings the amount of indirect radiation required to a small amount compared with the amount of air required when heated exclusively by indirect radiarion; besides, it is very difficult to obtain sufficient room within the walls to carry up hot-air flues to the rooms in the upper stories, as the upper story rooms are usually smaller and more numerous than those in the first story.

The best place for a direct radiator is under the window sills. Now this requires a low radiator, while the tendency is nowadays to lower window sills. We would recommend that direct radiators not exceeding twenty-three to twenty-four inches high from the floor be used. In designing a house the window sills should be kept not less than twenty-six inches from the floor. With low radiators they usually need to be long in order to make the requisite heating surface; this should also be looked after. It is well to keep this matter in mind in designing a building, and make your windows either wide or twin or triple windows, which afford room for the requisite length of radiators. It is not desirable usually to extend the radiators beyond the window, at least the ladies particulary object to it. If you cannot place the direct radiators under the window sills, then place them against exterior wall or as near as is practicable. If you heat the exposed sides of a room you will heat the whole of it.

Considerable trouble sometimes arises from the objections of the owners, and especially their wives, in placing the radiators, and sometimes practically objecting to radiators at all, or else putting them in such places as will not allow sufficient size of radiator to be put in. The difficulty is increased by the fact that they want the radiators low down. Less objection is usually made to a low radiator than to a high one; but a low radiator requires a greater length than a high one for the same amount of radiator service.

Some owners object to giving any sufficient space for a heating apparatus, and architects do not usually make proper and sufficient provision therefor.

The direct-indirect radiators should be the highest made (45 inches) so as to give ample radiator surface to heat the incoming fresh air and to take up as little floor space as is possible. Do not place them under windows, but in an unoccupied place in stair hall.

It is usually best to consider the first and second story stair halls as one first-story room, and treat it accordingly. Where the second-story hall is large, or much exposed, then a minor portion of the radiationmay be put in second story, otherwise put it all in first story.

Some years ago the manufactures were very apt to overrate the surface in the radiators. This is not much the case now, except with radiators which have what is termed extended surfaces, that is, solid projections on radiators. Now, while it may be admitted that these projectons are of some value, they are not nearly so valuable as is claimed for them. It is not known how valuable they are, but they are of no very great value and the only safe way is not to consider them at all. There is no mystery about measuring a radiator and the surface it contains. You can measure it with a tape line as you can measure any other surface.

Given sizes of pipe refer to their interior diameters. Boiler tubes are given by their exterior diameters. Although strictly speaking they may be a pipe, they are never called so.

There are various plans of boilers, both of wrought and of cast iron. Unless it is a very large residence it is usual to adopt some style of boiler already made and intended for house heating; they are usually of cast iron. There are a number of good ones, and it can only be said that you must use your best judgement in selecting the boiler you wish to adopt.

There should be some relation between the grate area and the surface of boiler exposed to the fire. The heat all comes from the coal which is burned on the grate. If you have your furnace large enough for thorough combustion the heating surface is apt to be sufficient. It is usual to have the heating surface fifteen times the grate area; but the surface in the boiler directly exposed to the fire is much more valuable for heating purposes than the surface of passages which are not exposed to the fire, but only to the smoke.

There is such a thing as getting your boiler too complicated. It is indispensable that all parts of your boiler can be readily gotten at and cleaned out.

Anthracite coal is almost universally used for heating house boilers east of the Mississippi river, and it is assumed that that fuel is to be used.

If the house is a large one, it may be better to use a wrought iron or steel boiler. For residence use, where the dimensions are sufficiently large to require such a boiler, perhaps what is known as the "saddle" and the "fire box" or "locomotive boiler" are the best.

Having ascertained the total square feet of radiation the required square inches of grate area can be taken from Table II.

Table II.

The required grate area can be taken from the following table, which gives the requisite square inches of grate area for each 100 square feet of radiator surface:

Total sq. ft. of Radiator Surface in building.	Sq. inches Grate Surface.	Total sq. ft. of Radiator Surface in building.	Sq. inches Grate Surface.	Total sq. ft. of Radiator Surface in building.	Sq. inches Grate Surface.
100	120	800	630	1 500	1 040
200	208	900	693	1 600	1 100
300	288	1 000	754	1 700	1 150
400	362	1 100	813	1 800	1 210
500	433	1 200	872	1 900	1 260
600	501	1 300	930	2 000	1 310
700	567	1 400	986		

Each heating apparatus should have an automatic temperature regulator on it. It adds to the comfort and saves fuel. There are various kinds of them, some of them are worked by electricity.

It is well to put a pop safety valve on all boilers, although it is not absolutely necessary with the open method of hot-water heating. A one-inch pop safety valve is usually sufficient.

A steam boiler must have a safety valve and pressure gauge.

A boiler grate should give facilities for the removal of clinkers.

In all heating apparatus containing water, provision should be made for emptying same by means of waste cocks which are placed at the lowest point of the apparatus and the water run off to sewer or other suitable place.

The size of the brick smoke flue, that is, its cross sectional area, should be one-fifth of the grate area. None should be less than 12 by 12 inches, and it is better to make them larger, make it 16 by 16 inches if practicable; this will be large enough for an ordinary residence. 12 by 16 inches is usually a fair size.

Some authors say that a flue may be too large. My experience is the other way, even with smoke stacks ten feet square inside.

Some cast-iron boilers have a small smoke outlet. This is a defect. In such cases it is all the more necessary to have a full size brick flue.*

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*Note: Some of the lecture has been omitted, but stands substaniially as delivered.

THE SEWERAGE OF THE WORLD'S COLUMBIAN EXPOSITION.*

By C. B. McClure, '91, and F. M. Spalding, '93, School of Civil Engineering.

The World's Columbian Exposition has complete systems of water-works and sewerage sufficient in capacity for a city of 400 000 inhabitants. Owing to the flatness of the site and to the cost of construction of a combined system of sewers, the sewerage is divided into three distinct systems: 1. Roof drains. 2. Storm water sewers. 3. Sanitary or ejector sewers. The different systems will be described in order.

ROOF DRAINS.

The roof drains, carrying the water from the roofs, have their outlet in the nearest water-way. In designing the size and grade of these sewers, the maximum rate of rainfall was taken at one inch per hour, all of which was considered to reach the sewer. The amount of water-way provided is probably not sufficient for heavy storms, and the sewers at such times will necessarily flow under a head, because the assumed rate of rainrall is often doubled and sometimes trebled in short, sharp storms In a permanent system this might prove a serious difficulty, but for the few heavy storms likely to occur during tht continuance of the Fair, there will probable be no trouble.

On the Mines and Mining Building there are 62 6-inch down spouts. The roof area of this building is 225 000 sq. ft., making a down spout for every 3 630 sq. ft. The Manufactures and Liberal Arts Building has 224 6-inch down spouts, or one down spout for every 5 930 sq. ft. of roof area. Down spouts are made of galvanized iron. Connection is made with the sewer by a 6-inch vitrified pipe leading directly from the bottom of down spout to the sewer. The total number of manholes in the roof drain system is 49.

The following table gives the total amount of vitrified sewer pipe used in the construction of roof drains:

Table Showing Length and Size of Pipe.

DIAMETER OF PIPE IN INCHES.	LENGTH IN FEET.	DIAMETER OF PIPE IN INCHES.	LENGTH IN FEET.
6	30,091	12	5,684
8	7,259	15	5,052
9	3,161	18	2,181
10	4,159		

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*The writers desire to express their obligation to F. H. Eno, '91, Asst. Engineer in the Water Supply and Sewerage Department of the World's Columbian Exposition, for valuable data used in the preparation of this article.

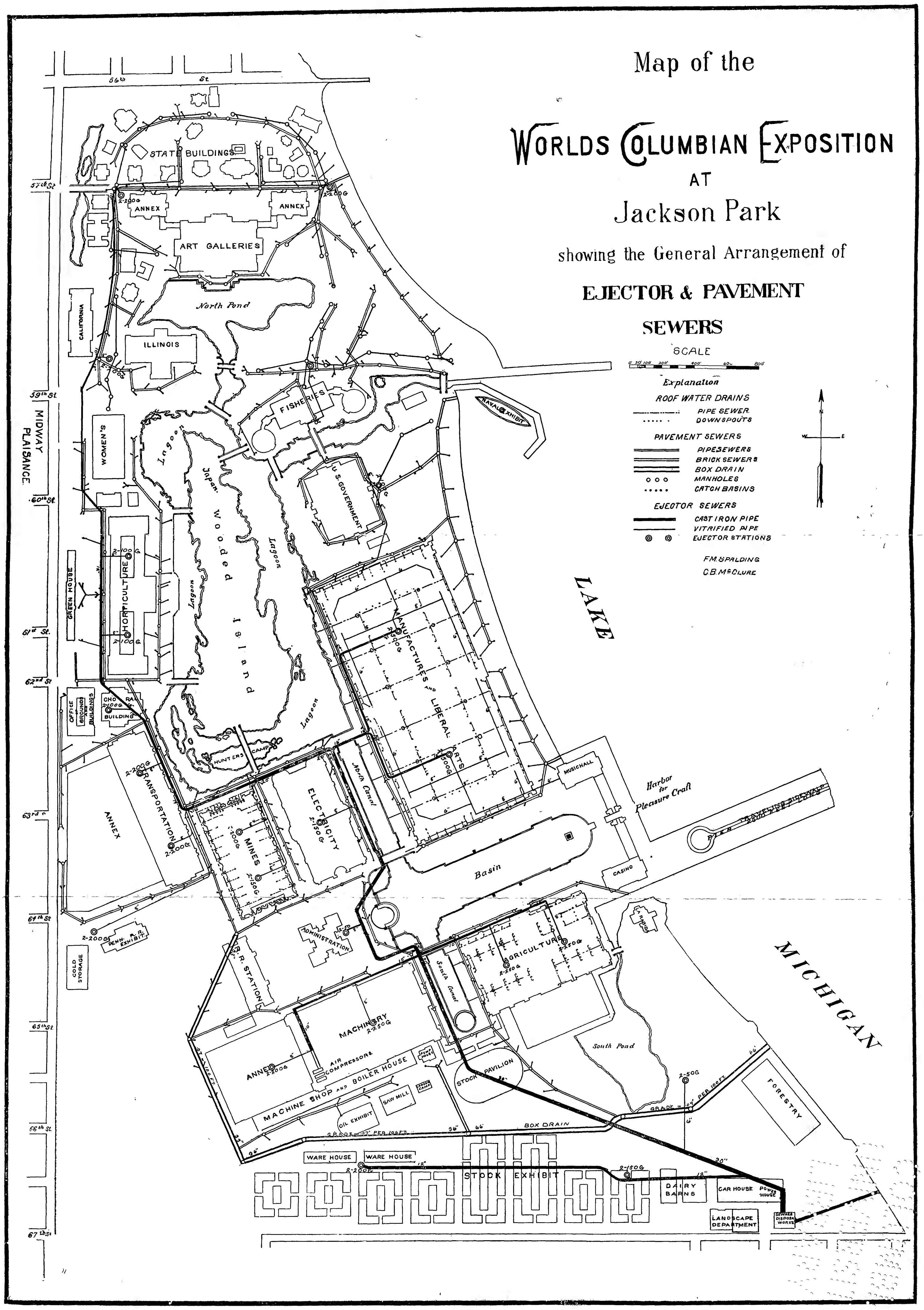
STORM WATER SEWERS.

The storm water sewers carry all water falling on the pavements, sidewalks and grass swards. It is very desirable to keep the water in lagoon, canals and basin free from filth and dirt of all kinds, and as the storm water sewers will carry a large quantity of paper, rags, peanut shells, etc., all pavement sewers discharge into the lake. For this drainage the grounds are divided into three districts: first, that draining the larger portion of the grounds including the area south of 59th street, and west and south of the lagoon and basin and having its outlet in the southern part of the grounds just north of Forestry Building; second, that draining the area around the Manufacturers Building and Government Building, and having its outlet east of the Government Building; third, all that portion of the grounds lying north of 59th street, and having its outlet at the head of 57th street. The location of the outlets, mains, man holes, catch basins and connections are shown on the accompanying map of the grounds.

The maximum rate of rainfall was taken at one inch per hour over the area drained, excluding the buildings, and it was assumed that all this rainfall reached the sewer. Kutter's formula, with n=.013, was used in the computations, and 2 feet per second was the minimum velocity allowed. This gives approximately the following minimum grades: 6-inch pipe, 1 in 450; 8-inch pipe, 1 in 250; 12-inch pipe, 1 in 450; 18-inch pipe, 1 in 850. At the time of this assumed flow the storm water sewers will flow under a head of 2 to 6 feet, but it is not probable that they will often be so heavily taxed.

The sewers consist of salt-glazed vitrified sewer pipe, brick sewers and box drains. Vitrified sewer pipe are used for lines varying from 6 to 18 inches in diameter. Brick sewers are used for lines varying from 18 to 36 inches in diameter. Above 36 inches box drains are used. The pipe used was first-class vitrified sewer pipe. Where excessive quicksand and water was encountered, deep-socket pipe was used, the joints being calked with oakum before cementing. Deep-socket pipe have a bell three inches deep, and the spigot and inside of bell are grooved or roughened to hold the oakum in place. The brick sewers are made of single or double rows of brick, according to the amount of water and quicksand encountered and to the depth of sewer below the surface.

The top and sides of the box-drain are constructed of pine, while



Map of the

WORLDS COLUMBIAN EXPOSITION

AT

Jackson Park

showing the General Arrangement of

EJECTOR & PAVEMENT SEWERS

the bottom is concrete. The sides consist of 3x9-inch tongue-and-grooved sheeting 9 feet long driven to the proper grade. Two inches from the top on the inner side of the sheeting 4x4-inch stringers were attached. 4x6 inch cross girders were laid on these stringers 2 feet apart. The girders were notched 4 inches to make them flush with the sheeting. The top is made of 2x6-inch matched plank securely nailed to the cross girders. The bottom is concrete 1 foot thick. The concrete consists of one part mortar to three parts broken stone. The mortar is composed of one part Portland cement to three parts of sand, both by measure. The radius of curvature of the bottom varies with the width of the sewer from 3.5 to 5.5 feet. The width of sewer varies from 3 to 5.5 feet. The depth at center varies from 3 ft. 6 in. to 3 ft. 9 in. The box drains were used on account of cheapness and ease of construction, and are well adapted to the temporary work for which they are designed.

Manholes on the pipe and brick sewers were put in 250 to 300 feet apart on the straight lines, and also at every change of direction of sewer. Manholes were built of brick, the walls being 8 inches thick laid in mortar, and were finished at pavement grade with a cast-iron suburban manhole cover. The cover is somewhat lighter than that used in the business portion of the city. The total number of manholes on the pavement sewers is 316.

Catch basins are usually placed in pairs one on either side of the roadway, the distance between pairs varying from 150 to 300 feet. The catch basins are made of a 3-ft. section of 18-inch vitrified sewer pipe with an 8-inch spur moulded on at an angle of 34 degrees. Pipes are set on a bed of concrete 6 inches thick and are finished on the inside with a coat of neat cement mortar, troweled smooth. Catch basins are finished at pavement grade with a cast-iron grated cover. The cover is set in cement mortar. The connections with the sewer are made with curved and straight pipe. The total number of catch basins of sewer pipe is 473, and in addition there are 50 brick catch basins.

In order to beautify the interior water-ways and the lake shore all outlet pipes are required to be 1.5 feet below datum. Datum is taken as the average level of Lake Michigan for several years, according to the records of the city of Chicago. The elevation of the surface of the ground varies from 5 to 11 feet above datum. The depth of sewers below the surface varies from 3.5 to 12 feet. The average cut was about 5.5 feet. The ground was excavated in open trenches to the necessary depth, with a width two feet wider than the exterior diameter of the sewer. The work was carefully back-filled, the earth being rammed under and around the pipe, the remainder of the trench being filled without ramming. Pipe were laid in bottoms of 25 feet under a line in perfect conformity with lines and levels given.

The contractor furnished all pipe specials, cement, yarn, etc., and did all excavating and back-filling. Nearly all the ground was sandy; some gravelly quicksand, necessitating the use of sheeting, was met at all points below 1.0 foot above datum.

The following tables give the total length of sewers constructed and the price per lineal foot:

TABLE GIVING TOTAL LENGTH AND COST PER LINEAL FOOT OF VITRIFIED PIPE SEWER.

DIAMETER OF PIPE IN INCHES.	ORDINARY SOCKET.		DEEP SOCKET.	
	TOTAL LENGTH IN FEET.	PRICE PER LINEAL FOOT.	TOTAL LENGTH IN FEET.	PRICE PER LINEAL FOOT.
4	6354			
5	1056			
6	32021	$0.275	412	$0.29
8	40668	0.315	5451	0.36
9	1614	0.40	496	0.50
10	7910	0.53	1584	0.60
12	8594	0.65	2980	0.72
15	3430	0.80	3284	0.90
18	4992	0.98	523	1.08

TABLE GIVING TOTAL LENGTH AND COST PER LINEAL FOOT FOR BRICK SEWERS AND BOX DRAINS.

DIAMETER OF SEWER IN INCHES.	TOTAL LENGTH IN FEET.		PRICE PER LINEAL FOOT.
	DOUBLE RING BRICK SEWER.	BOX DRAINS.	
21	570		$1.48
22	558		1.55
24	2241		1.62
27	730		1.80
28	550		1.84
34	289		2.25
36	2105		2.65
33		5690	2.85
66		2072	5.15

Manholes cost $15.00 apiece and catch basins $11.20. The total length of pavement sewers is 26.1 miles, and the total cost of this system is approximately $98,860.

SANITARY SEWERS.

The third and most important system is the ejector or sanitary system of sewers for the removal of sewage proper. There are 16 buildings provided for the purposes of the Exhibition, which vary in size from 1 to 30.5 acres. Besides these there are 40 or more smaller buildings built by the different states and by foreign countries. There are in all 150 acres of buildings for which sanitary sewers must be provided.

A daily attendance of 150 000 with a probable maximum of 400 000 is anticipated. For the accommodation and convenience of this multitude there are numerous toilet rooms and lavatories with a total of 3 500 water closets and 1 000 urinals, and with wash basins in proportion. The maximum attendance of 400 000 will occur between the hours of 7 A. M. and 11 P. M., during 16 hours of a day; in other words this would be a rate of attendance of 600 000 people per 24 hours. As the maximum quantity of sewage expected is 8 500 000 gallons, which is the capacity of the disposal works, the maximum quantity of sewage per capita expected is then 8 500 000÷600 000=14 gallons per 24 hours.

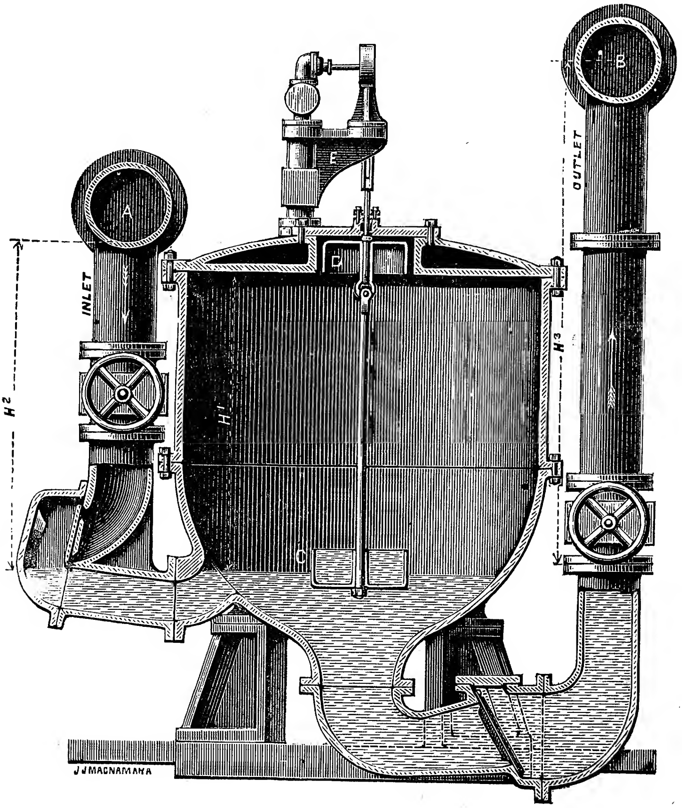
Any system of collection of sewage by gravitation sewers alone would necessitate very light grades owing to the flatness of the site, it varying from 5 feet to 11 feet above Lake Michigan. If the combined system were used, the sewers would necessarily be large and expensive. It was therefore decided to put in a separate system of sewers for the removal of the sewage proper and to pump the sewage to the purification works in the southeast part of the grounds. The Shone system was adopted to do this work.

The ejectors are placed in pairs in stations so constructed that the sewers will flow by gravity into the ejectors.

The accompanying cut gives a section of the ordinary Shone Pneumatic Ejector. The action is as follows:

The sewage flows in through the inlet pipe and gradually fills the large chamber till the sewage reaches the under side of the bell D. This of course confines the air and as the flow continues the pressure of this confined air lifts the spindle and this opens the compressed air valve E. The compressed air thus admitted presses on the surface of the sewage and forces it out through the outlet pipe. The sewage cannot flow back up the inlet pipe because the inlet valve is forced shut as soon as the compressed air is admitted. The sewage continues to pass out of the ejector till the cup C is no longer supported; then the weight of the sewage contained therein is sufficient to pull down the spindle and close the air valve E. The air in the ejectors is then allowed to exhaust to atmospheric pressure and the whole action is repeated. At the junction of the discharge pipes from the two ejectors there is placed an air chamber similar to those seen on ordinary steam pumps; this chamber is designed to act as a cushion for the outrushing sewage and to prevent as much as possible the shock or water ram that would otherwise result. The air chamber is 2 feet high and 10 inches in diameter.

There are 26 ejector stations located over the grounds, the majority being beneath the buildings, some under the grass plots and some under the roadways. These stations consist of a wooden or brick pit in which are two ejectors, one ejector being held in reserve and used only in case of emergency. When the stations are inside the buildings the pit is built of 4x6-inch matched sheeting



and is 10 to 11 feet deep and 11.5 feet in diameter, with 2 feet of concrete in the bottom. The pits are designed to be water tight. When the stations are outside the buildings the pits are constructed with a 12-inch brick wall and have 18 inches of concrete in the bottom. The depth of the pit is 11 feet and the width is 11.5 feet; the top is covered with two layers of 6x6-inch timbers laid crosswise. The capacity of the ejectors varies from 60 to 600 gallons per minute and their united capacity is 11 400 gallons per minute or 17 000 000 gallons per 24 hours. In other words both ejectors in each pit will have to discharge once a minute.

There are 3 miles of gravitation collecting sewers running into the ejectors. These sewers are either 6 or 8-inch deep-socket vitrified pipe, except when laid inside the buildings, when wrought iron pipe is used. The minimum grade for the 6-inch is 1 in 150 and for the 8-inch 1 in 250.

The air compressing plant for operating the ejectors is situated in Machinery Hall. Its location is shown on the accompanying map. It was designed and built by the Norwalk Iron Works of South Norwalk, Conn. The plant consists of three compound air compressors and one double compound air compressor. These compressors are unique in that they have two air cylinders with a cooling chamber between. The initial pressure is made in a large air cylinder and the final compression is made in a small air cylinder. The capacity of the plant is 4 537 cubic feet of free air per minute.

There are 5 miles of air pipe leading from this plant to the different ejector stations, varying in diameter from 2 to 10 inches. Those laid under buildings and those less than 4 inches in diameter are wrought iron screw pipe; all the remaining pipes are cast iron. The average depth of covering is 4.5 feet. The line was tested to stand 90 pounds pressure per square inch without loss.

From the ejectors there are 4.8 miles of cast iron discharge pipe leading to the sewage disposal works. These sewers are shown in black full lines on the map. The size of pipe varies from 6 to 30 inches. "The maximum lift from the discharging level of the lowest ejector to the top of the precipitating tank is 67.6 feet and the total head to be pumped against when the maximum estimated amount of sewage is being discharged is 107.8 feet. A pressure of 47 pounds per square inch will be maintained in the air mains, and each ejector will be provided with reducing valves so that just the necessary pressure will be used."

The contractor was paid a certain price per foot for the pipe laid. Profiles were furnished of the pipe lines, and when the pipes were for any reason laid deeper, an extra allowance was made for the deeper cut. All specials and stop gates were put in as required by the engineer. The pipe was all furnished by the Exposition Company.

COST OF PIPE LAYING.

KIND OF MATERIAL.	DIAM.	COST PER FOOT.	KIND OF MATERIAL.	DIAM.	COST PER FOOT.
Wrought Iron	2 Inch.	$ .08	Cast Iron	12 Inch.	$ .27
“ “	3 “	.09	“ “	14 “	.43
“ “	4 “	.10	“ “	16 “	.45
Cast “	4 “	.13	“ “	20 “	.60
“ “	6 “	.17	“ “	24 “	.65
“ “	8 “	.20	“ “	30 “	.85
“ “	10 “	.22	“ “	36 “	.90

Contractor furnished lead, yarn, etc. $20 per 1 000 feet B. M. was allowed for timber left in the trench by order of the engineer.

COST OF ENTIRE EJECTOR SYSTEM.

Discharge Pipes:	*a.* Cast iron pipe and valves	$ 45 000
	b. Wrought iron pipe	20 000
	c. Laying	15 000
Vitrified sewers		8 000
Wrought iron feed sewers		20 000
Ejector pits		13 000
Ejectors		67 000
Total cost		$188 000

These figures are approximate; roughly, $200 000 was expended on the sanitary sewers.

SEWAGE CLEANSING WORKS.

The sewage cleansing works are located in the southeast corner of the grounds. The plant consists of four large vertical precipitating tanks and a central stand pipe through which the sewage is forced to the top of the precipitating tanks. The precipitating tanks are 32 feet in diameter and 50 feet high, the bottom 18 feet built conical in form. In the center of each precipitating tank there is a fixed vertical tube 6 feet in diameter and 30 feet deep, having at its base 8 radiating arms extending to the sides of the tank. The central stand pipe is 60 feet high and 36 inches in diameter. Built around the top of the stand pipe is a large iron drum 16 feet in diameter and 11 feet high. The top of the drum is covered with a grating which acts as a screen for the sewage as it flows over the top of the stand pipe. The sides of the drum are tapped at four points by 14-inch pipes which conduct the sewage to the vertical tubes in the precipitating tanks.

Supported on an iron frame work on the top of the tanks are tubs of chemicals mixed ready for use. Aluminum sulphate and lime will probably be used, although copperas may be chosen. The chemicals enter the sewage in the 14-inch pipe, the aluminum sulphate entering first just above a mixer which thoroughly mixes this chemical with the sewage. A little farther along the lime is added. The sewage then flows down the vertical tubes fixed in the precipitating tanks and is distributed over the entire tank by the radiating arms; the solid matter settles to the bottom while the liquid slowly rises to the top in the large annular space surrounding the central tube. The settling solid matter acts as a filter for the rising liquid and in this way the highest efficiency is gotten out of the chemicals. Near the top of the tanks several wooden troughs are rigidly held, and as the liquid rises it overflows into these, and thence into a larger trough which empties into an 8-inch cast iron down spout. In this last trough is placed a weir for measuring the quantity of sewage. There are two down spouts, each serving two tanks. The two spouts are connected at their base with a pipe which conducts the purified water to the lake.

At the base of the precipitating tank are valves which may be opened; and the head in the tank is sufficient to force the sludge out and into either of three vertical cylindrical tanks. When one of these tanks is full the source is cut off and compressed air at 100 lbs. pressure forces the sludge into the sludge press and presses it into 50 rings or cakes, 1½ inches by 3 feet in diameter. There are small drip holes along the bottom of the press so that the water can run out. These cakes are then carted out to the Garbage Crematory and burned. There are two of these sludge presses; they will press from 12 to 15 tons of sludge per day.

It is designed to have all four precipitating tanks in operation at once. Their capacity will be 6 000 gallons per minute or 8 640 000 gallons per 24 hours. The cost of chemicals will be about $5 per 1 000 000 gallons of sewage, depending on the character of sewage, difficulty in procuring chemicals, etc. From 6 to 8 men will be required to operate the plant when running at its maximum rate.

The entire cleansing plant is contained in a building 100x125 feet, and cost $25 000, not including the building.

THE NEW MERCURY COLUMN AT THE UNIVERSITY OF ILLINOIS.

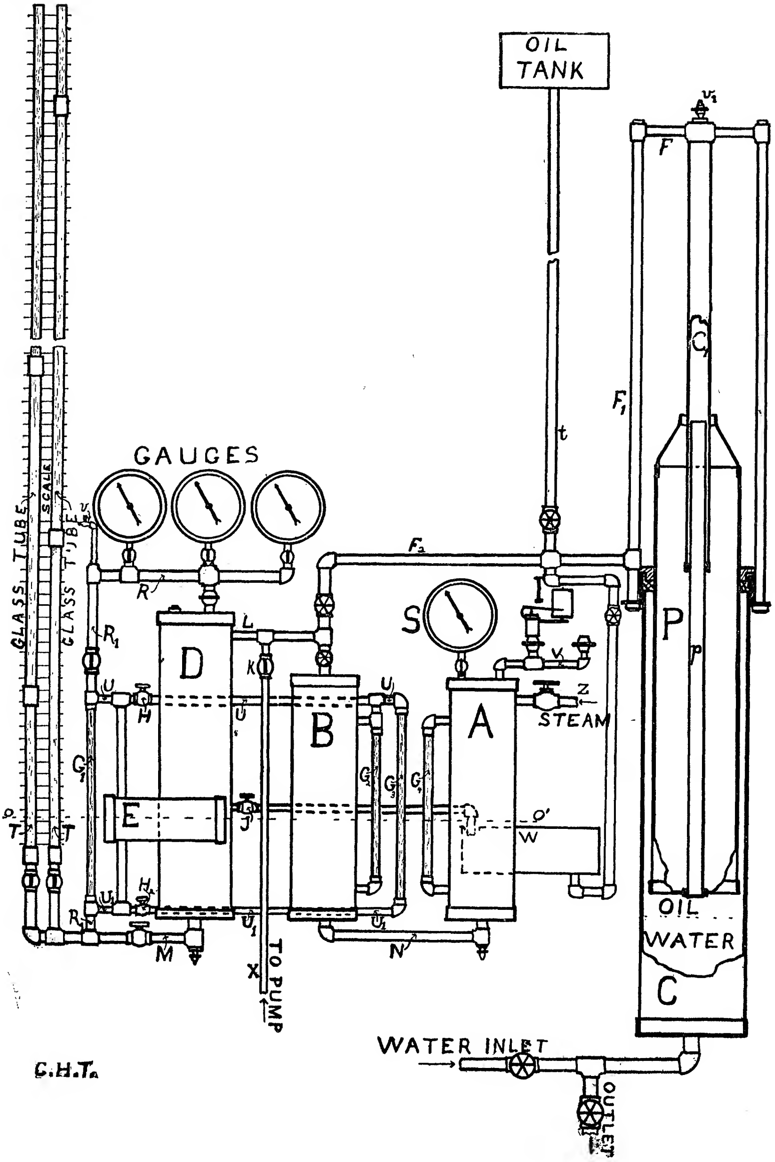
By Chas. H. Trego, '94, School of Elecrical Engineering.

Two objects were in view in improving the mercury column as described below. First, to substitute for the slow and irregular action of the hand pump, some automatic means for producing a continuous and easily regulated pressure. Second, to provide an attachment for testing steam engine indicators by direct steam pressure measured by the mercury column. Both these ends have been accomplished with entire success, a very rapid, or an exceedingly gradual rise in pressure up to 200 pounds can be automatically obtained with perfect ease, and the indicator attachments are found to be in every way an improvement on the means commonly used. The apparatus was designed by Professor Scribner, and has been constructed under his supervision.

In using it the weight of a column of mercury is balanced against the elasticity of the spring of the steam gauge or indicator, and the pressure thus acting on the spring, is computed from the measured height of the mercury.

Pressure can be applied by the small hand pump, which forces oil in through the pipe X; also by steam through the pipe Z, or by the hydraulic apparatus shown at C, the working of which will be described later. In whichever way the pressure is applied, oil is forced through the pipe L into the cylinder D. The lower part of this cylinder contains mercury, which the oil forces out through the pipe M into the glass tubes TT forming the column. The height to which the mercury rises depends upon the pressure applied to the oil. The tubes TT are made of heavy glass, in six foot lengths, and behind them is a scale divided into inches and running the full length of the tubes. The joints in the tubes are at different levels, so that the height of the mercury can always be read in one tube when it is hidden by the joint in the other. To find the pressure due to the mercury, its height in inches above the zero mark, is multiplied by the weight of a cubic inch of mercury at the observed temperature. This gives the pressure in pounds per square inch. The weight of a cubic inch of mercury at 60° F. is 0.4 911 74 pounds.

The gauges to be tested are screwed to cocks on the pipe R. This pipe is connected to the vertical pipe R_1, which runs downward and connects to the pipe M containg mercury under pressure; a glass



New Mercury Column at University of Illinois.

gauge tube G_1 forms part of the pipe R_1. Two pipes, U and U_1, branch off from R_1 just above and below the glass part G_1, and from these, two other pipes connect to the top and bottom of the horizontal cylinder E; the pipes U and U_1 continue on to the right to another glass gauge tube G_3. Thus the two glass gauges G_1 and G_3, and the cylinder E are so connected that when they are filled up to about the middle of E with mercury, and from that point to the top of R_1 with oil, the line between the oil and mercury will be at the same level in all three. The cock in the pipe from the top of the cylinder D to the pipe R is only opened to fill R, G_1, E, etc., with oil at the start, and *must be kept closed* at all other times. The cocks H and H_2 can be closed so as to shut off the tube G_3, and this is done in testing gauges. If the vent cock V in the top of the pipe R_1 be opened, and oil forced into the cylinder D, mercury will rise into the tubes T T and G_1 and the cylinder E. At first it will rise rapidly, but after it gets up to about the center of E it will rise very slowly, on account of the large volume of E which has to be filled for a slight rise in height. The vent cock V in top of R_1 is now closed, so that the only way for the mercury to rise further in G_1 and E is by forcing oil into, and thus expanding, the springs in the gauges. Of course this expansion will require very little oil, and as a proportionally large quantity of oil is forced out of E for a slight rise of the mercury, the level of the mercury in G_2 and E will now be practically constant. It will be readiy seen that the cylinder E acts just as if it were a large bulb blown in G_1 at that level.

The zero of the scale is taken at the level at which the mercury in T T stood, when the vent cock V was closed. Since this cock is at the same level as the centers of the gauges, the level of the mercury in T T will be just enough above that in E, as shown by the gauge glass G_1, to balance the height of the oil up to the gauge centers.

To test steam engine indicators accurately, the test must be made with the indicator at the temperature it has when in use. Many schemes have been devised for doing this, but the best way is to use steam in the indicator at the desired pressure. When this apparatus is used for testing indicators the cylinders A and B are used in connection with the part already described. These cylinders are connected at the bottom by the pipe N, and each has a glass tube, G_2 and G_4, on its side to show the level of the water with which each is about half filled. The space in A above the water is filled with steam, which is admitted through the pipe Z, and that above the water in B is filled with oil. When the cock H is opened the oil in B can pass through a connection between the glass tube G_2 and the pipe U, and on through into G_1 and E. The valve in the pipe from the top of B to L *must be kept closed* when testing indicators.

The indicator I is attached to the pipe V, at the top of the cylinder A, a steam gauge S being also connected to this cylinder. This gauge must be known to agree with one on the pipe R. Before testing the indicator the water levels in A and B, as shown by their gauge glasses, must be made to agree with each other and with the mercury level in E, which should be half way up in the latter and can be read both at G_1 and G_3. For this purpose the cocks H and H_2, in the pipes V and U_1, are opened and mercury in G_3 and the water in G_2 and G_4 are adjusted to the same level O–O′, near the center line of the cyclinder E. The cocks H and H_2 are now closed and oil is forced into D and steam let into A until the desired pressure is reached. When the gauges on A and R read alike the cocks and H and H_2 are opened, and any slight difference in pressure is at once equalized. If the levels of the water and mercury in G_2, G_3 and G_4 have changed slightly, they can be adjusted. A line is now drawn on the card by the indicator, and the mercury column is read from O as zero; the pressures thus represented are afterward compared to find the accuracy of the indicator. This process is to be repeated, at intervals of a few pounds pressure throughout the whole range of the indicator spring.

A great advantage of this process is that no constant need be added or substracted from the height of the mercury column as read on the scale, provided, of course, the zero of the scale is at the same level as the mercury in G_2. This comes from the fact the water in A and B, and the mercury in E, all have the same level, as shown by the gauge glasses G_2, G_3 and G_4. From this the water pressures in A and B balance each other, the oil in B balances that in E, and the mercury in E balance that below the same level in TT. The weight of the oil in the pipe R_1 will make a little difference in the reading of the guages, but this does not enter into the result in testing indicators. As previously explained, it is allowed for in testing gauges.

These tests, for both gauges and indicators, should be made both with increasing and decreasing pressures to eliminate any friction errors in the instruments tested.

To use steam for producing pressure on the oil, the cocks H and H_2 must be closed, and the valve in the top of B opened. Steam is now let into A, and it forces water over into B; this drives the oil up and out of B and over into D, after which the effect is the same as before. This method is suitable for gauge testing only.

The hydraulic apparatus shown at C is generally used for producing the pressure. All parts of it are within easy reach, and as the pressure can be regulated so nicely it is probably as convenient an arrangement as could be devised. It is nearly all made of iron pipe and pipe fittings, and about the only machine work done was in turning the large plunger P, which works in the cylinder C. Inside of P, and fastened to it at the bottom, is the small plunger P, which works in the small cylinder C_1. The areas of these plungers are as 20 to 1. The large cylinder can be filled with water from the city mains through the pipe in the bottom. Two valves in this pipe let the water flow in or out as desired. There are a few inches of oil on the water in this cylinder to keep the plunger from rusting when it is left out of the water. The small cylinder C_1 is filled with oil, as are the pipes F, F_1, F_2, etc., that open into it. When the large plunger is moved up the small plunger rises in the cylinder C_1, and forces the oil in C_1 out through the pipes F, F_1, etc., over into the cylinder D. The pipes F_1 and F form part of the frame of the machine. The large plunger is weighted to bring it down again when the water outlet in the bottom of the cylinder is opened. The pipe t connects F_2 with a small oil tank in the second story, so that the oil can be forced from the machine up into it, or the machine can be filled from it, as desired.

From the lower part of the oil level in D, the pipe J, runs over to the open reservoir W. This affords a convenient method of lowering the oil pressure slightly without lowering the plunger. In case it is desired to lower the mercury column without lowering the plunger, the oil can be drawn off through the pipe J; this oil can afterwards be drawn up into the cylinder C_1 by opening the valve in the pipe from W to F_2 and lowering the plunger. Drip cocks are placed on the lowest parts of cylinders and pipes, so that they can be entirely emptied, and vent cocks on the highest parts let the air out when the cylinders are filled again.

EQUIVALENT CIRCULAR CURVES FOR RAILROAD TURNOUTS.

BY R. C. VIAL, '93, CIVII ENGINEERING COURSE.

In location work curves for branches or sidings are often run from a main tangent track. It is often desirable to know what equivalent circular curve may be used, instead of locating the turnout proper, such that when the turnout is put in, the equivalent circular curve will be tangent to the center line of turnout at the heel of frog, the center line of turnout being straight from toe to heel of frog. Fig. 1 shows the parts of the turnout and the equivalent circular curve, the latter being represented by the light curved line, the turnout by the heavier lines. There is of course a different curve for each frog number. The scale of the drawing is greatly distorted in order to show the parts more clearly.

It may be well before considering the formulas for the curve to consider the leads used in computing this equivalent curve. Very frequently leads are used such as to require more or less fudging in putting in turnouts, even a simple turnout from a straight track. This is needless. The writer has computed the parts of a turnout from a straight track for a split switch, (Table I,) and has had some of the leads tested in actual practice. The road on which they have been tried is the Belt Railway of Chicago. The parts fit together nicely and the results have been very satisfactory.

The assumptions on which the computations were based are: First, that the switch rails are straight—15 feet long—T H Fig. 1. This is the common length. Second, that the curved rail of the turnout is tangent to the switch rail, T H, at the heel, H, of the switch and is tangent to the frog rail at the the toe of the frog, (P. T. in Fig. 1.) Third, that the frog rails are straight from the toe to the heel of frog, and that the distance B—P. T. is 4 feet. There is no uniformity in this length but 4 feet is an average for all frogs. Fourth, that the throw, or the distance between the gauge sides of the main and turnout rails at the heel of the switch, H, is $5\frac{1}{2}$ inches. The switch angle, S, is the angle between the gauge sides of the main track rail and the straight switch rail. It may be seen then that

$$\text{Sin } S = \frac{\text{throw of switch}}{\text{length of switch rail.}}$$

For a $5\frac{1}{2}$ in. throw and 15 ft. switch rail, $S = 1°45'$, and this has been used in computing Table I.

TABLE I.

Parts of Standard Turnout for Split Switch.

Frog Number.	Frog Angle.	Length of Switch.	Throw of Switch.	Lead.	Turnout Curve.	
					Radius.	Degree.
		FEET.	INCHES.	FEET.	FEET.	
4	14° 15′	15.0	5.5	42.10	105.4	
5	11° 25′	15.0	5.5	48.88	176.6	32° 54′
6	9° 32′	15.0	5.5	55.26	266.7	21° 37′
7	8° 10′	15.0	5.5	61.39	378.2	15° 12′
8	7° 09′	15.0	5.5	67.18	510.9	11° 14′
9	6° 22′	15.0	5.5	72.61	665.3	8° 37′
10	5° 44′	15.0	5.5	77.85	846.4	6° 46′

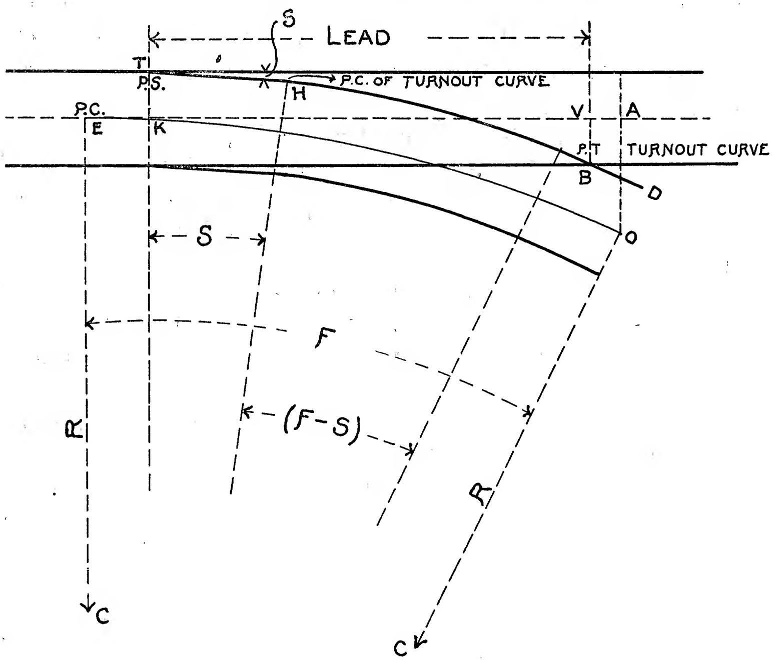


FIG. 1.

The equivalent curve is indicated in Fig. 1 by the light curved center line, O E; the turnout by heavier lines, the center line of turnout not being shown.

The formulas for the parts of the equivalent curve are easily deduced, hence their demonstration is not given in full.

Let F=the frog angle,
S=the switch angle,
G=the gauge of track,
R=the radius of the equivalent curve,
P. S., the point of switch when located,
P. C., the point of curve of equivalent circular curve,
H, the heel of switch,
B, the theoretical point of frog,
P. T., the toe of frog,
and D the heel of frog.

By trigonometry $AO = R \text{ vers } F$,

$$\text{But } AO = BD \sin F + \tfrac{1}{2} G\,(1 + \cos F)$$

Substituting and solving for R,

$$R = BD \cot \tfrac{1}{2} F + \tfrac{1}{2} G \cot^2 \tfrac{1}{2} F \qquad (1)$$

$$EA = R \sin F \qquad (2)$$

$$\text{Chord } OE = 2\, R \sin \tfrac{1}{2} F$$

Substituting for R

$$OE = 2\, BD \cos \tfrac{1}{2} F + G \cot^2 \tfrac{1}{2} F \sin \tfrac{1}{2} F \qquad (3)$$

$EK = (R + \tfrac{1}{2} G) \sin F - BD \cos F - KV$ (the lead for that frog angle) (4)

Using the values of the lead, etc., from Table I, and the above formulas, Table II has been computed. This gives the parts of the equivalent curve for the frog numbers indicated, using BD equal to the following values:

Frog Number	BD
4	5.0 ft.
5	5.0 ft.
6	5.0 ft.
7	6.5 ft.
8	8.5 ft.
9	9.0 ft.
10	9.25 ft.

There is no uniform length for BD. These values represent good practice.

The column headed EK is the most important as it gives the distance to be measured from the P. C. of the equivalent curve to locate the point of switch, P. S., for the frog number for which the curve is equivalent.

The degree of curve is given to the nearest 5 minutes.

TABLE II.—PARTS OF EQUIVALENT CURVES.

FROG NUMBER.	FROG ANGLE.	EQUIVALENT CURVE.				
		RADIUS.	DEGREE.	CHORD O E	P.C. TO P.S. E K	A O
		FEET.		FEET.	FEET.	FEET.
4	14° 15′	189.3	30° 40′	47.00	0.21	5.87
5	11° 25′	285.3	20° 10′	56.53	3.18	5.65
6	9° 32′	398.8	14° 30′	66.23	6.21	5.50
7	8° 10′	549.0	10° 30′	77.96	11.48	5.61
8	7° 09′	739.3	7° 45′	91.70	16.73	5.75
9	6° 22′	926.0	6° 10′	101.86	21.50	5.69
10	5° 44′	1130.2	5° 0′	113.00	27.21	5.62

To illustrate, suppose it is desired to put in a turnout using a No. 9 frog. In the table (Table II.) opposite No. 9 it is seen that a 6° 10′ curve is the equivalent curve; i.e., a 6° 10′ curve starting from the main line 21.5 ft. back of the point of switch has the same direction at O, Fig. 1, that the No. 9 frog has at that point, or in other words a 6° 10′ curve is tangent to the center line of the turnout at O.

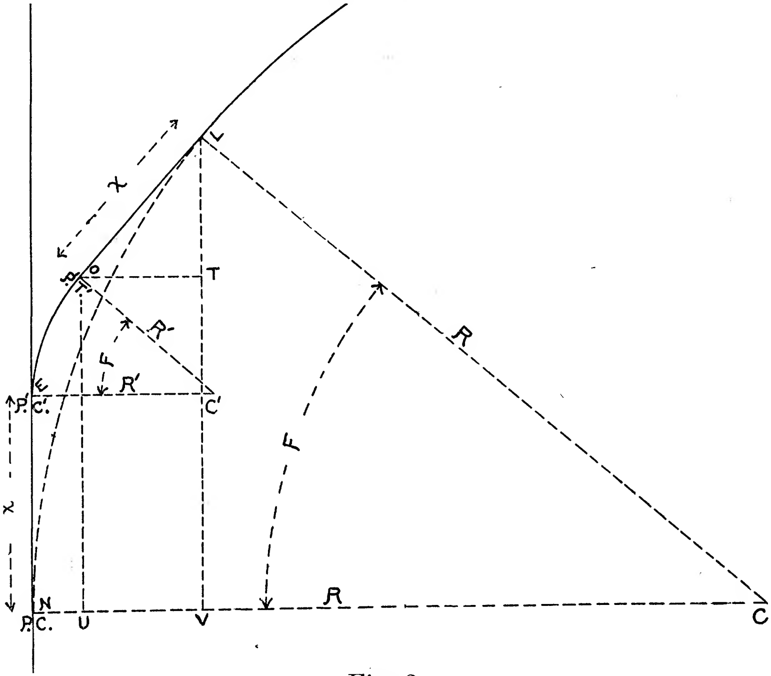


Fig. 2.

This point, O, may or may not be a point of compound curve as is desired.

The following is an important application of the equivalent curve.

To replace a curve by a standard turnout and short tangent. Fig. 2 represents the center lines only.

P. C. is the point of curve of the main curve whose radius is known. P′. C′. is the point of curve of the equivalent curve for the specified frog number. X is the connecting tangent. The two portions marked X are equal of course.

By trigonometry $X = R \sin F - R' \sin F - X \cos F$.

Solving for X and substituting, $X = \dfrac{(R - R') \sin F}{2 \cos^2 \frac{1}{2} F}$

R′ is known, (see Table II), hence X is known. Then all that is to be done in locating the point of switch is to measure from the P. C. of the main curve a distance equal to the sum of (1) the value of X for that curve and frog number and (2) the value of E K from Table II for the same frog number.

In Table III the values of X are given. In Table IV are the distances to be used in locating the point of switch from the P. C. of the main curve. The point of switch having been located the frog point is determined as usual.

To find the distance from the point of frog to the end of the connecting tangent, add to the length of tangent beyond the end of frog the length of the straight frog rail from point to heel of frog. Each of these two quantities is found in Table III.

To illustrate, supposing that a 6° curve has been run from a tangent track and that it is desired to put in a turnout to connect the main track with the curve using a No. 7 frog. Looking in Table III opposite No. 7 frog and under the column headed 6° we find that it will require 29.0 feet of tangent track beyond the frog to connect with the curve. But to locate the point of switch for this No. 7 frog turning out into a 6° curve we look in Table IV opposite No. 7 frog in the column headed 6° and find that the point of switch for this frog is 40.5 feet from the P. C. of the curve as located.

TABLE III.—Length of Connecting Tangent.

Frog Number.	Frog. Angle.	Straight frog rail point to heel of frog.	Degree of Main Curve.								
			2°	3°	4°	5°	6°	7°	8°	9°	10°
		FT.	FT.	FT.	FT.	FT.	FT.	FT.	FT.	FT.	FT.
4	14° 15′	5.0	334.5	215.2	155.9	119.6	90.8	78.7	65.9	56.0	48.1
5	11° 25′	5.0	258.0	162.5	114.7	86.1	67.0	53.4	43.2	35.2	28.8
6	9° 32′	5.0	205.5	126.0	86.2	54.3	46.4	35.0	26.5	19.9	14.6
7	8° 10′	6.5	169.3	97.2	63.1	42.6	29.0	19.3	12.0	6.3	1.8
8	7° 09′	8.5	132.8	73.2	43.3	25.4	13.5	5.0	...	...	...
9	6° 22′	9.0	107.7	54.7	28.2	12.2	1.6	...	...	...	...
10	5° 44′	9.25	86.7	39.0	15.1	0.8	...	...	...	...	...

TABLE IV.—Distance from P. C. of Main Curve to Point of Switch.

Frog Number.	Frog Angle.	Degree of Main Curve.								
		2°	3°	4°	5°	6°	7°	8°	9°	10°
		FT.	FT.	FT.	FT.	FT.	FT.	FT.	FT.	FT.
4	14° 15′	334.7	215.4	156.1	119.8	91.0	78.9	66.1	56.2	48.3
5	11° 25′	261.2	165.7	117.9	89.3	70 3	56.6	46.4	38.4	32.0
6	9° 32′	211.7	132.15	92.35	60.5	52.6	41.2	32.7	26.1	20.8
7	8° 10′	180.8	108.7	74.6	54.1	40.5	30.8	23.5	17.8	13.3
8	7° 09′	149.55	89.9	60.0	42.1	30.2	21.7	...	...	...
9	6° 22′	129.2	76.2	49.7	33.7	23.1	...	...	...	...
10	5° 44′	113.9	66.2	42.3	28.0	...	...	...	...	...

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John B. Tscharner was born in Okawville, Illinois, April 15th, 1866. His boyhood days were spent there. In 1885 he entered the preparatory department of the University of Illinois, from which institution he was graduated in the civil engineering course in 1890. In the summer vacation of 1889 he was employed by the Union Pacific R. R., with headquarters at Cheyenne. After his graduation he returned to the same work, but soon left on account of the climate and went to Waco, Texas, where he was general superintendent of the Waco Light and Water Co. He was rapidly rising in his profession when without warning he had several severe hemorrhages of the lungs in May, 1892. He was taken to Colorado Springs at once, and for some time seemed to gain strength; in March, however, he began to fail, and died of hemorrhage April 22, 1893.

Mr. Tscharner was a very bright, active, business like man. His business ability was recognized while in college by his election as business manager of the first Technograph, and his rapid promotion after leaving the University shows the esteem in which he was held by his employers. He was an honorable, upright, promising man, with the best of habits and inclinations; a man of many warm friends, and one who was yearly becoming more and more an honor to his profession and to the University of Illinois.

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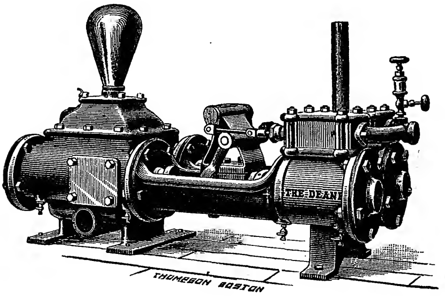

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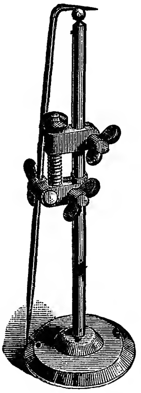

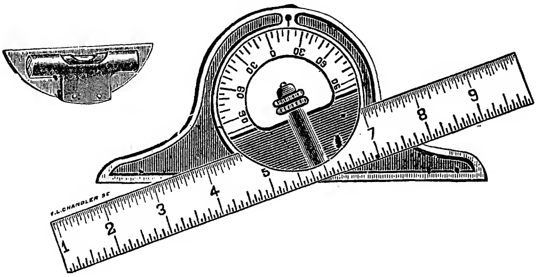

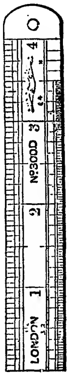

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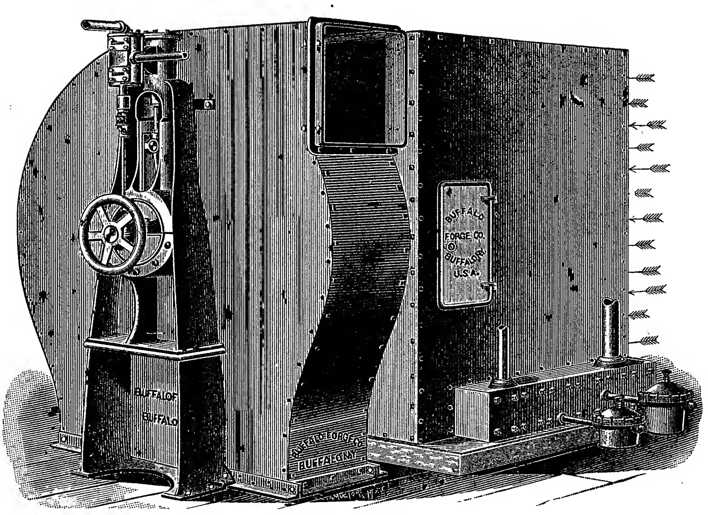

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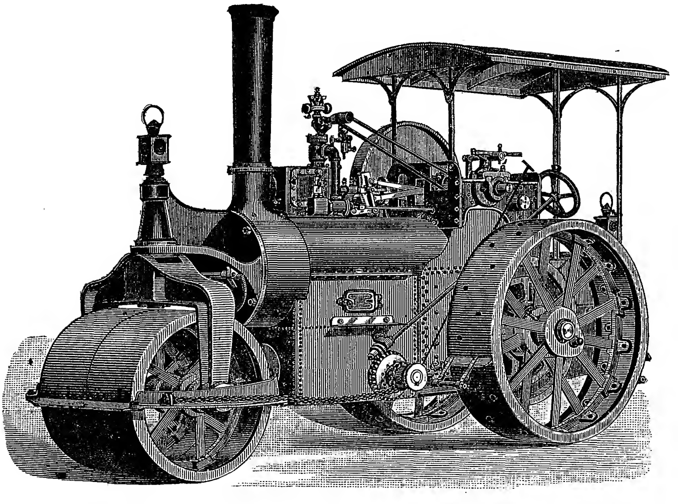

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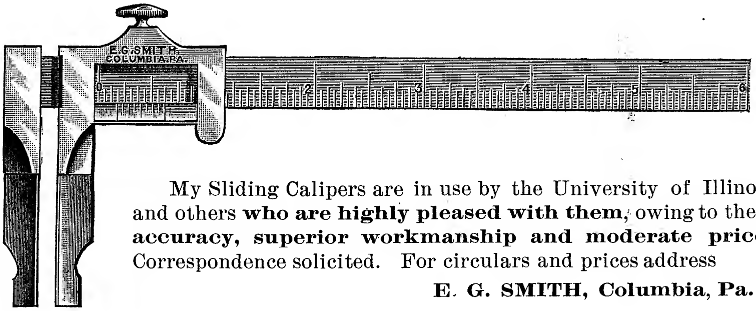

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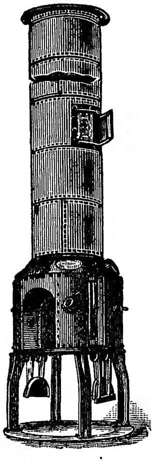

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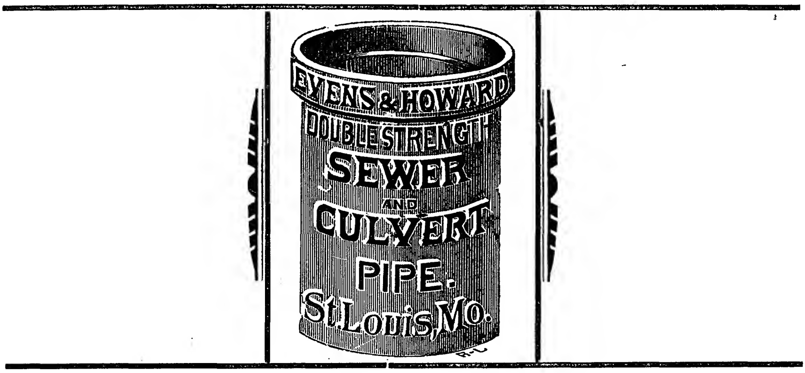

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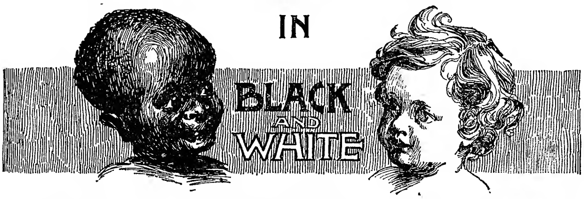

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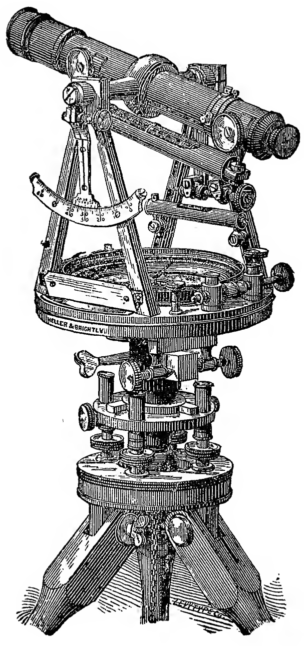

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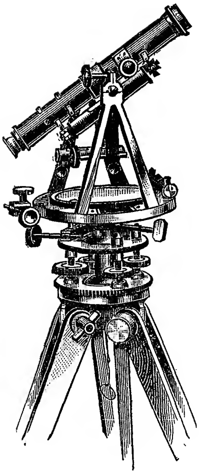

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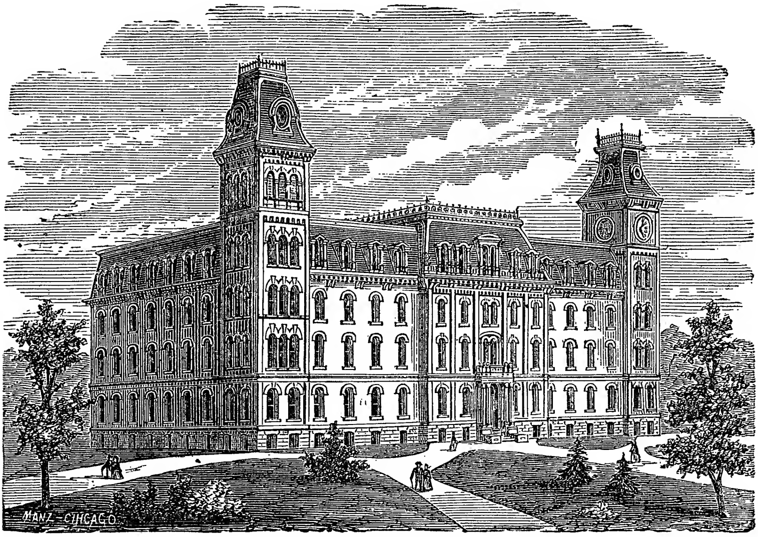
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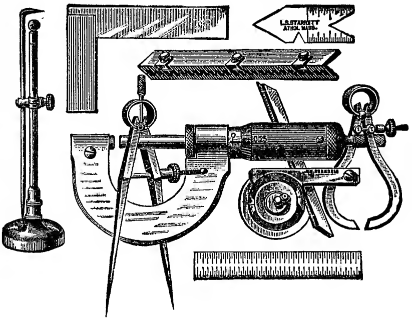

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